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CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

SEPTEMBER, 1952. Vol. XLVII, No. 9

FORTY-SEVENTH YEAR OF PUBLICATION

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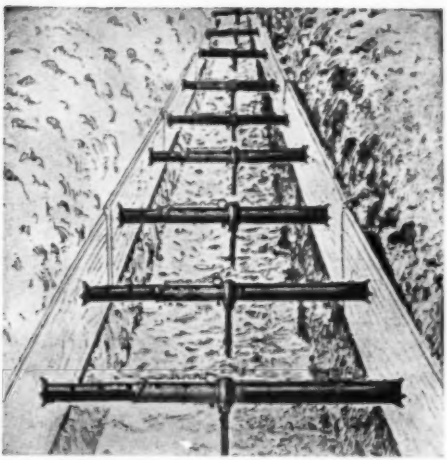

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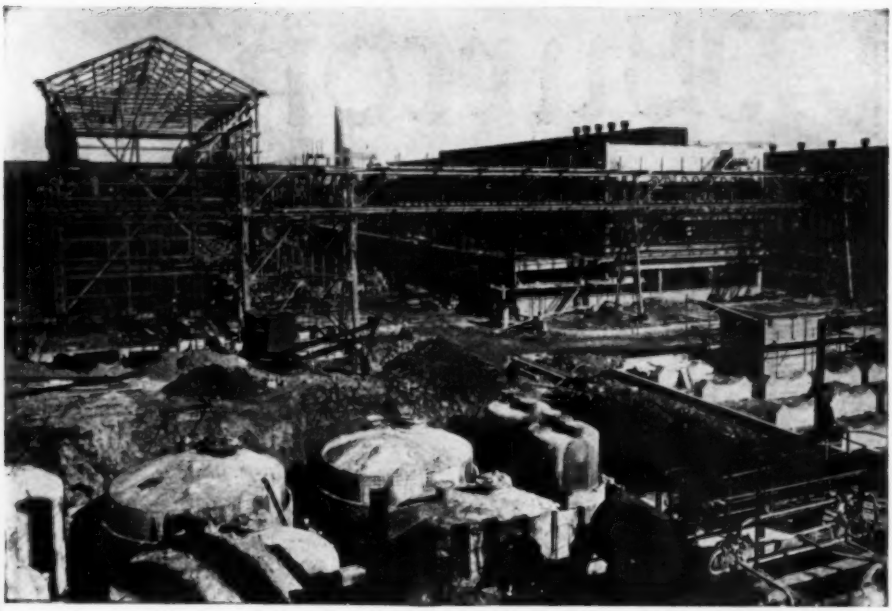

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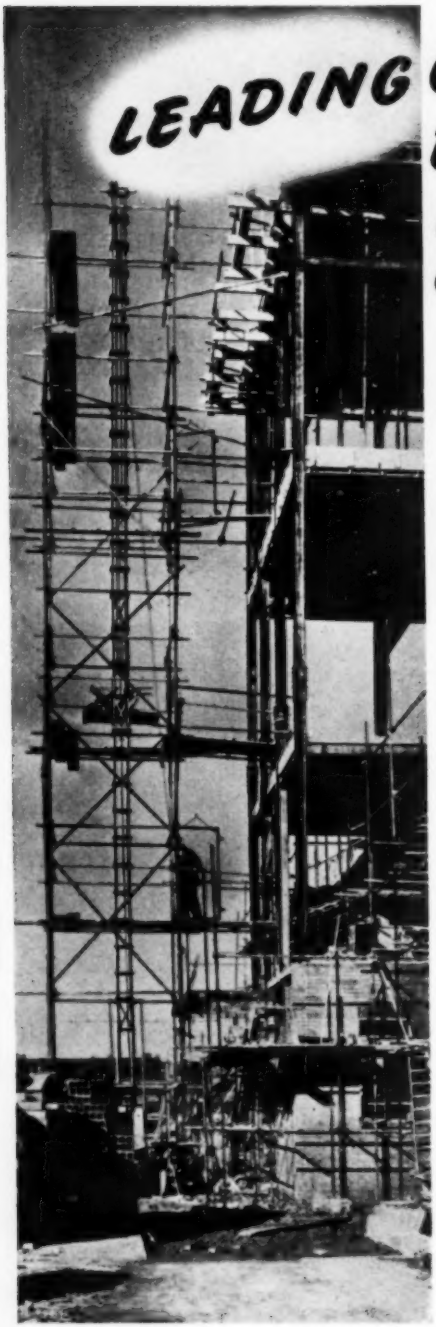

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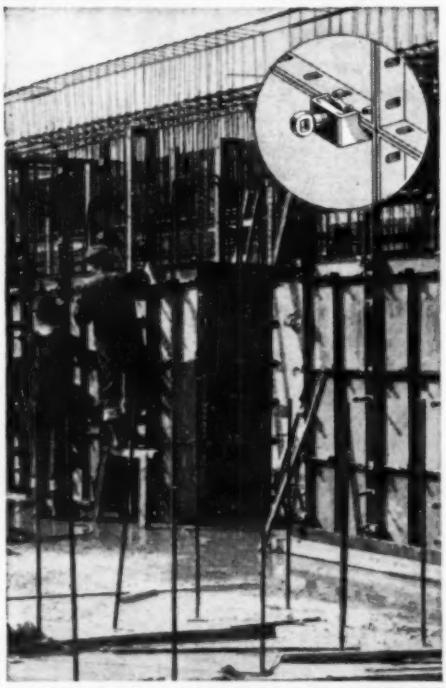

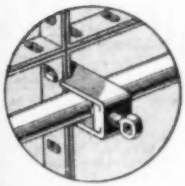

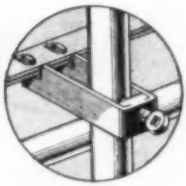

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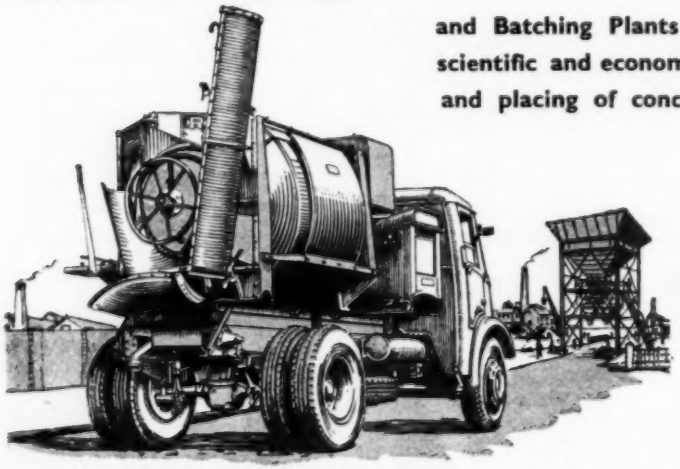

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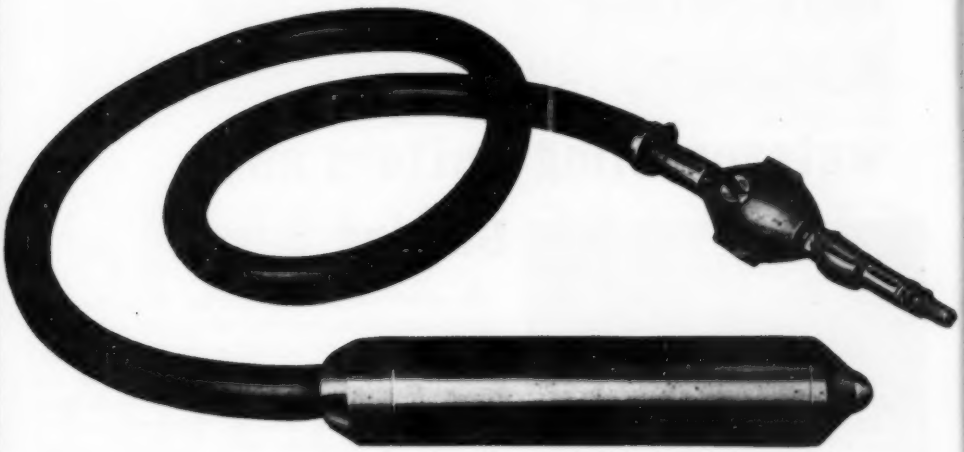

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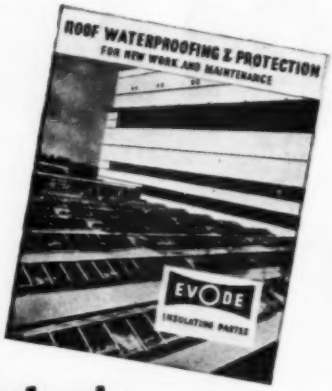

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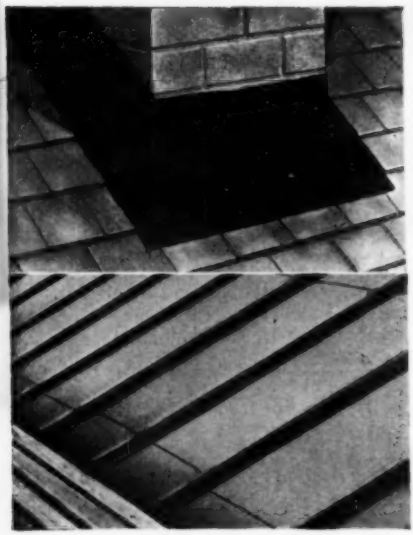

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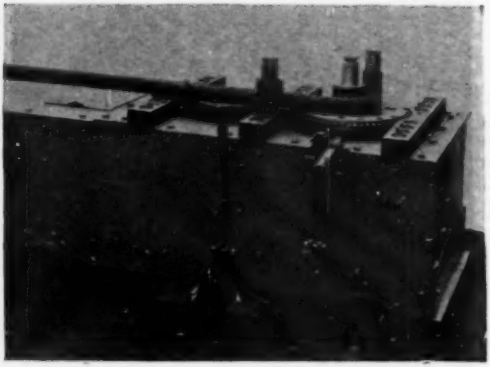

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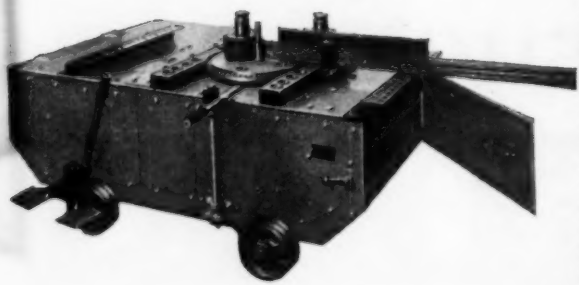

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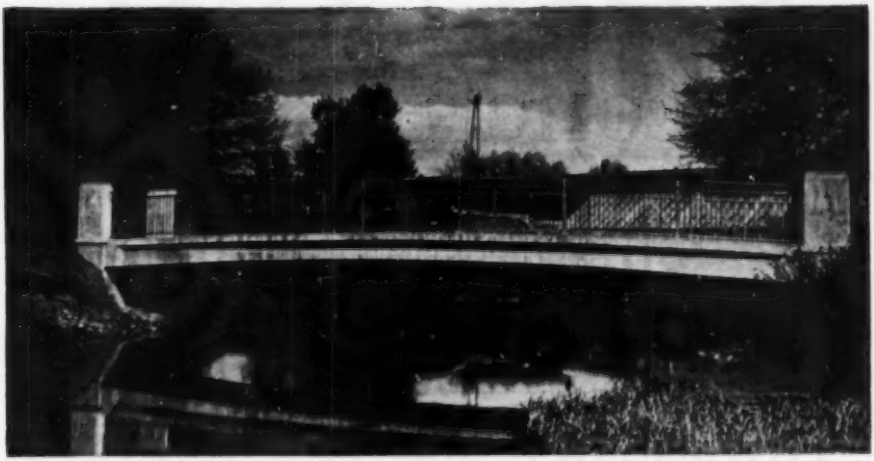

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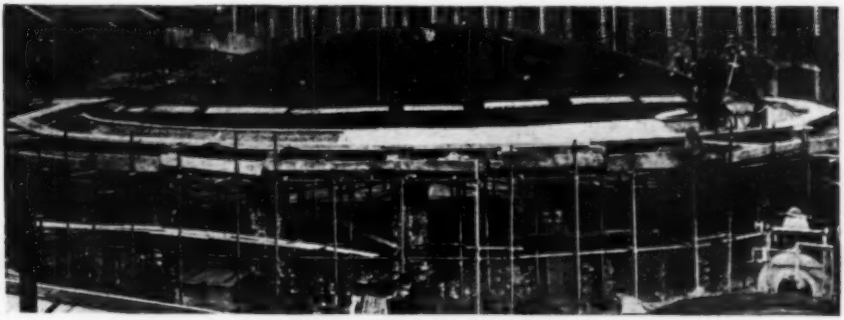

SHELL ROOF CONSTRUCTION

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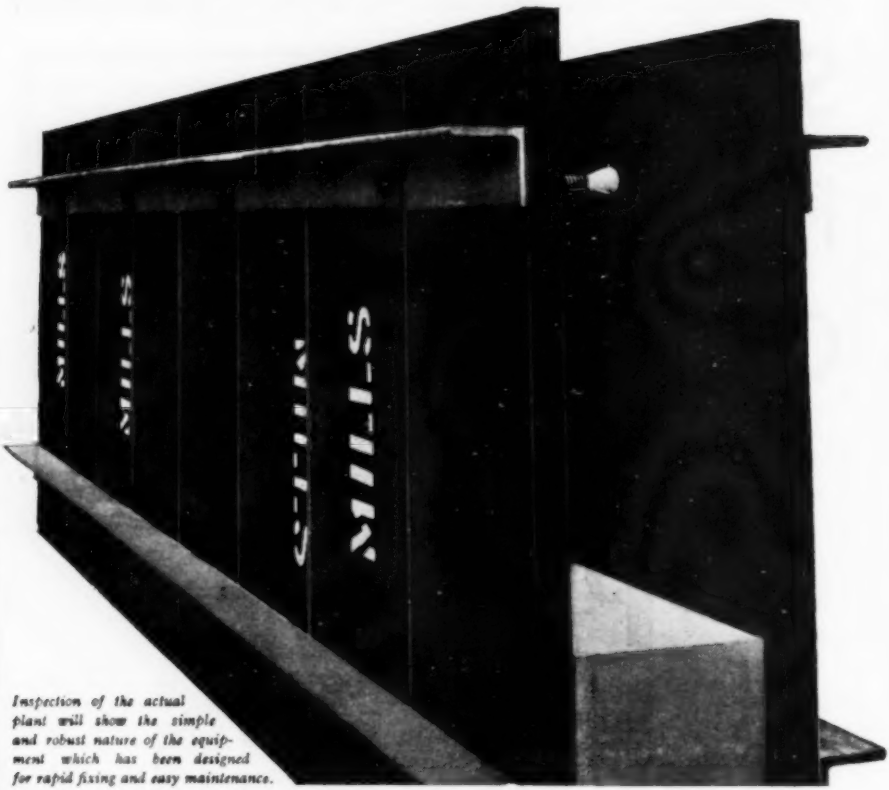

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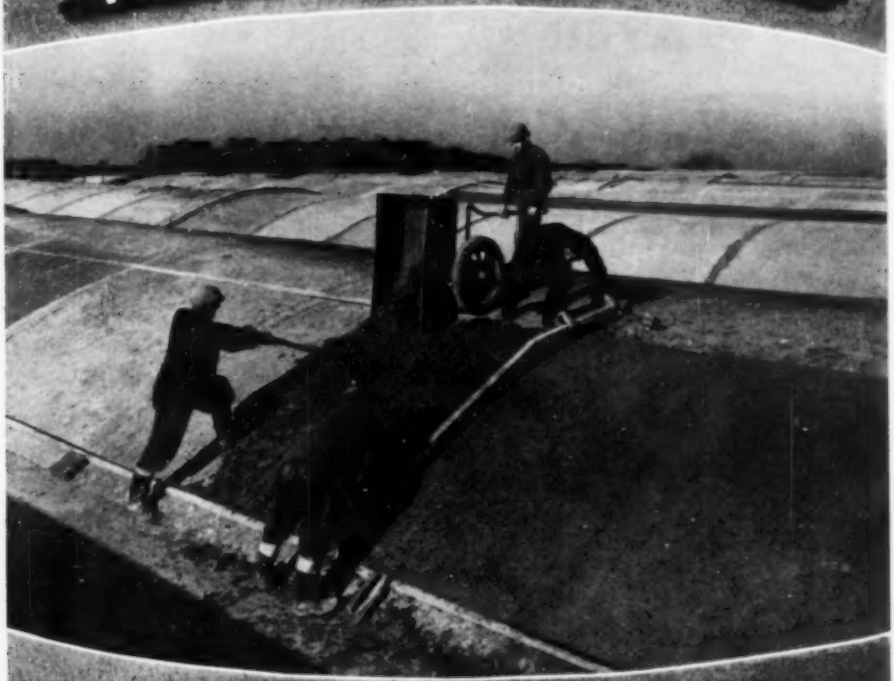

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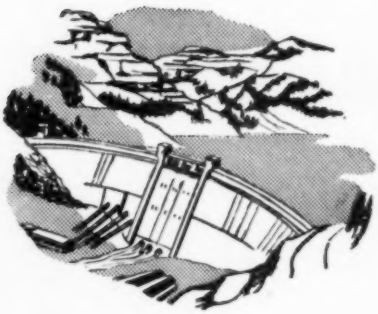

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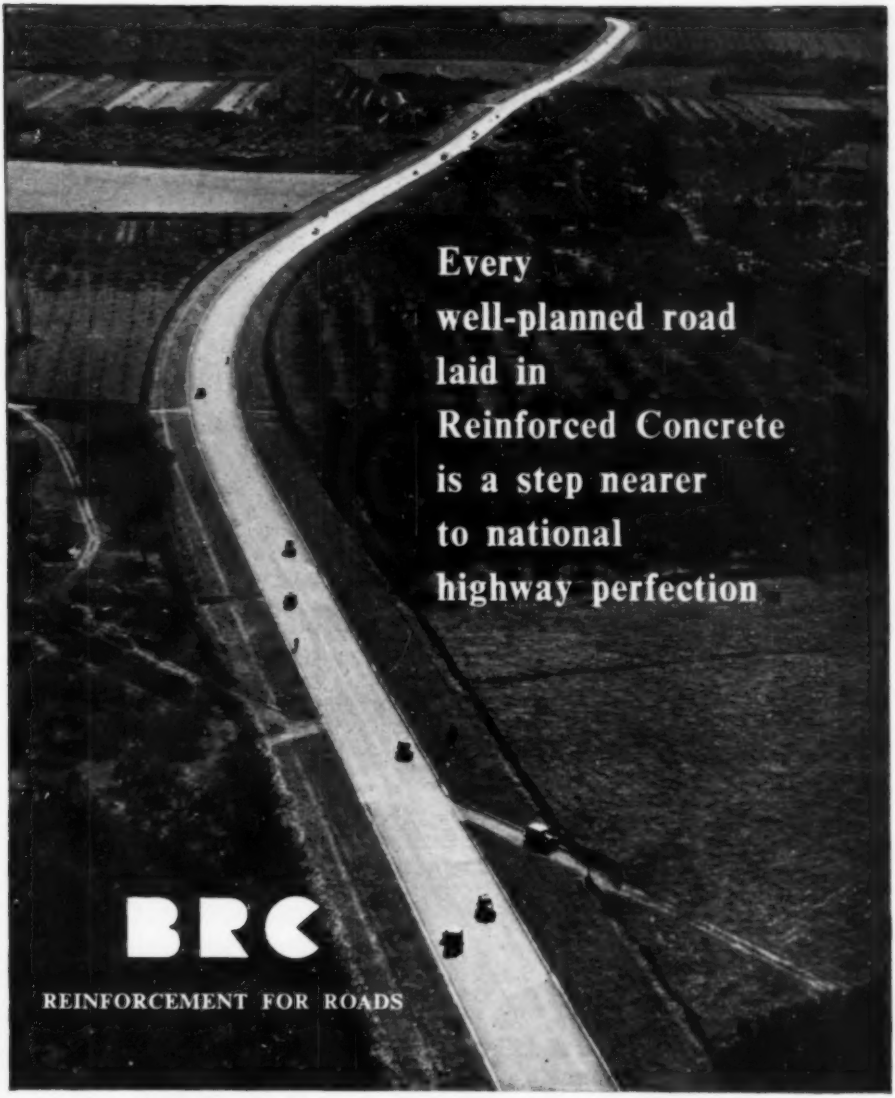

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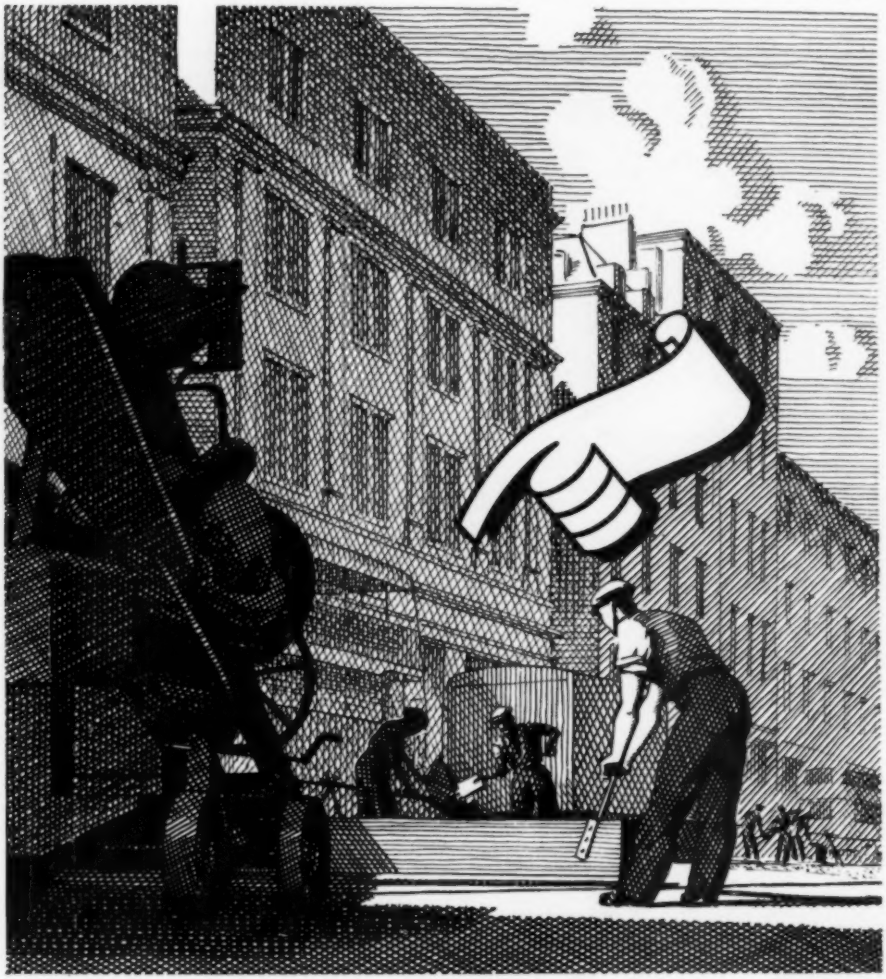

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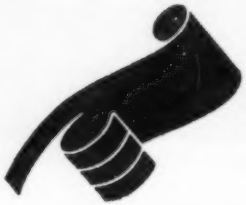

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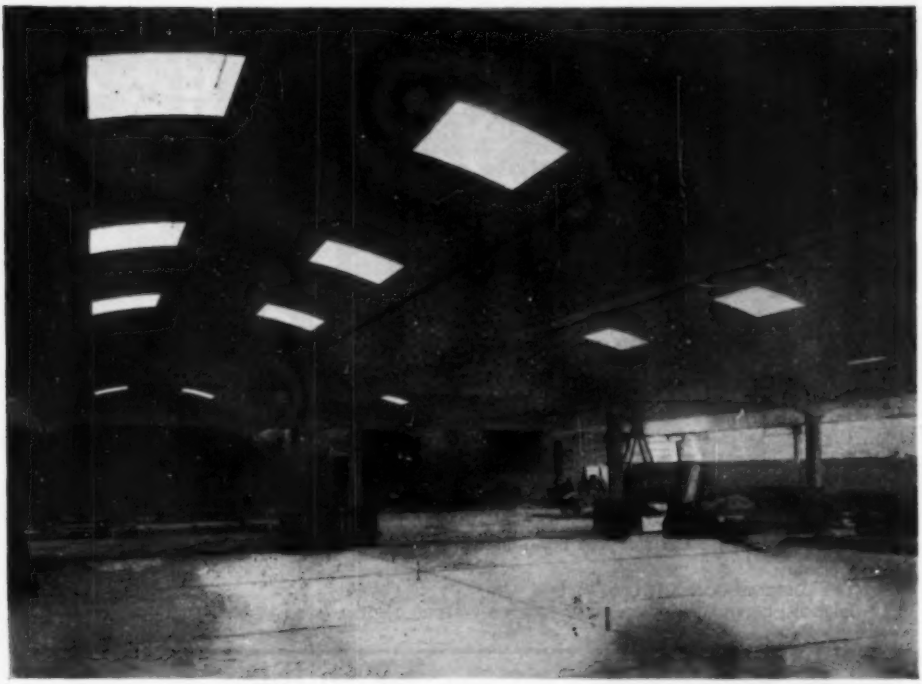

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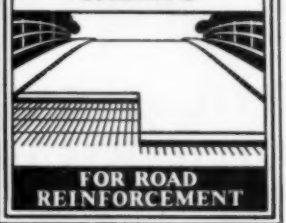

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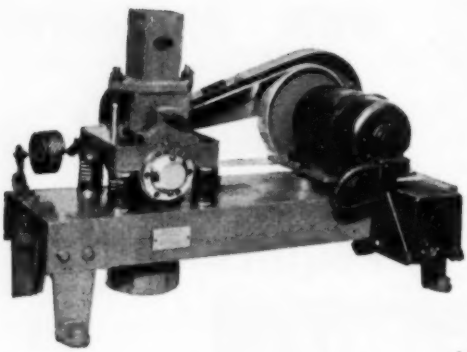

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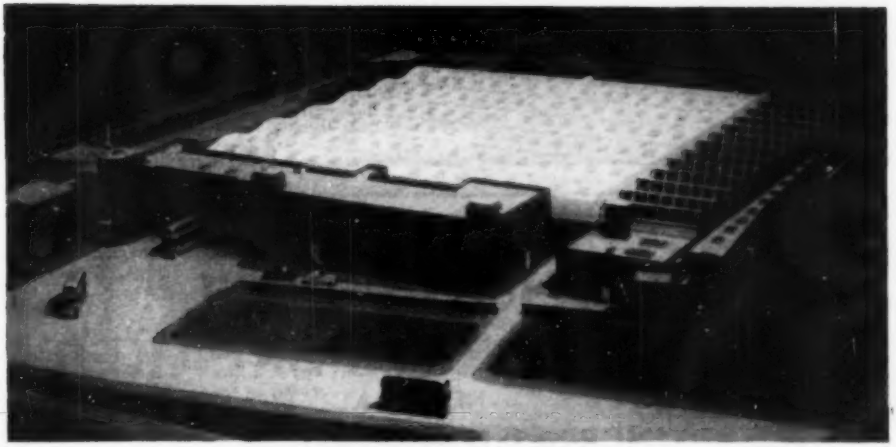

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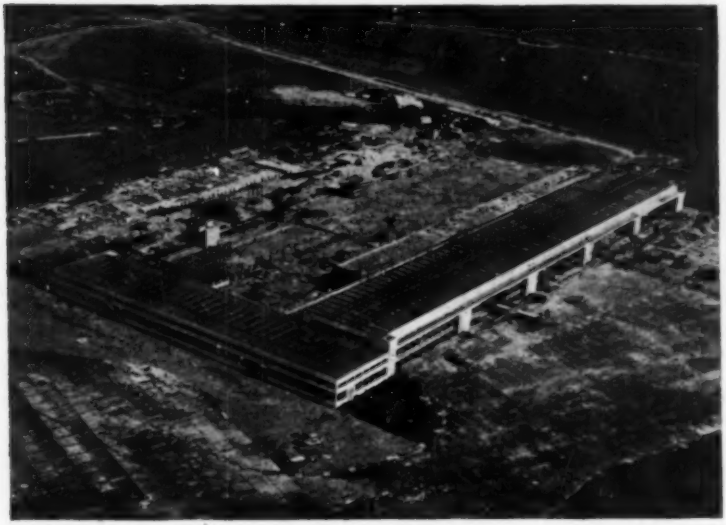

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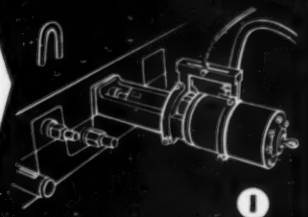

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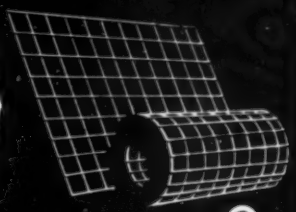

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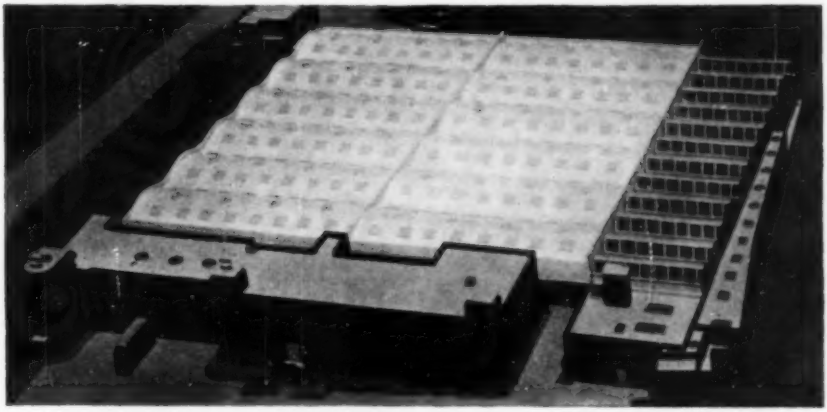

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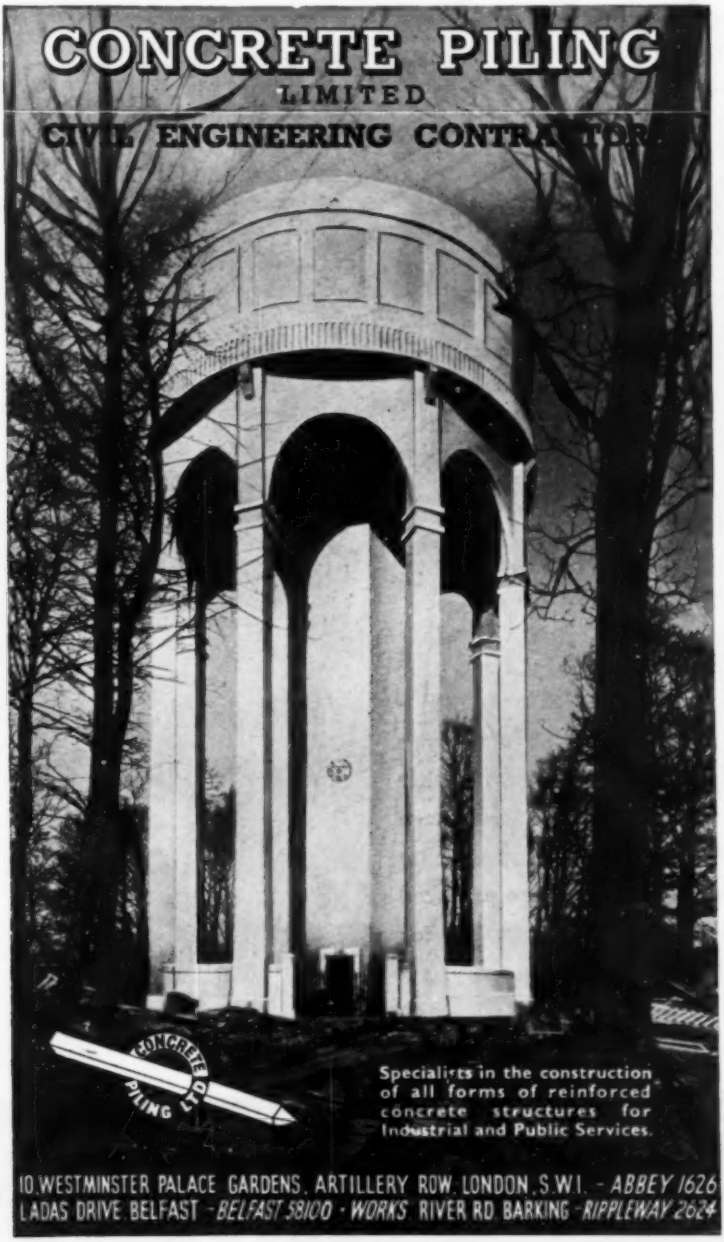
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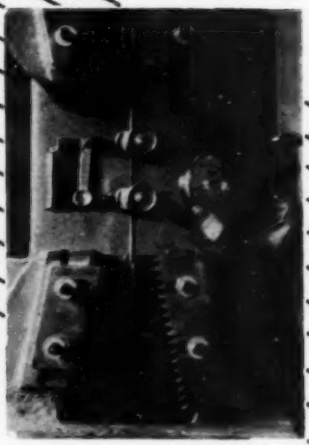

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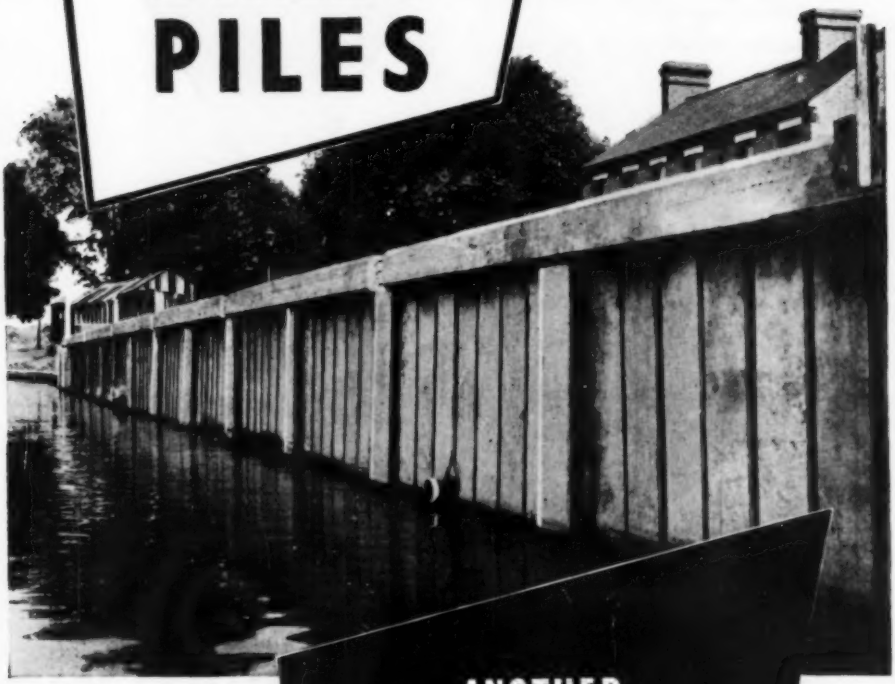

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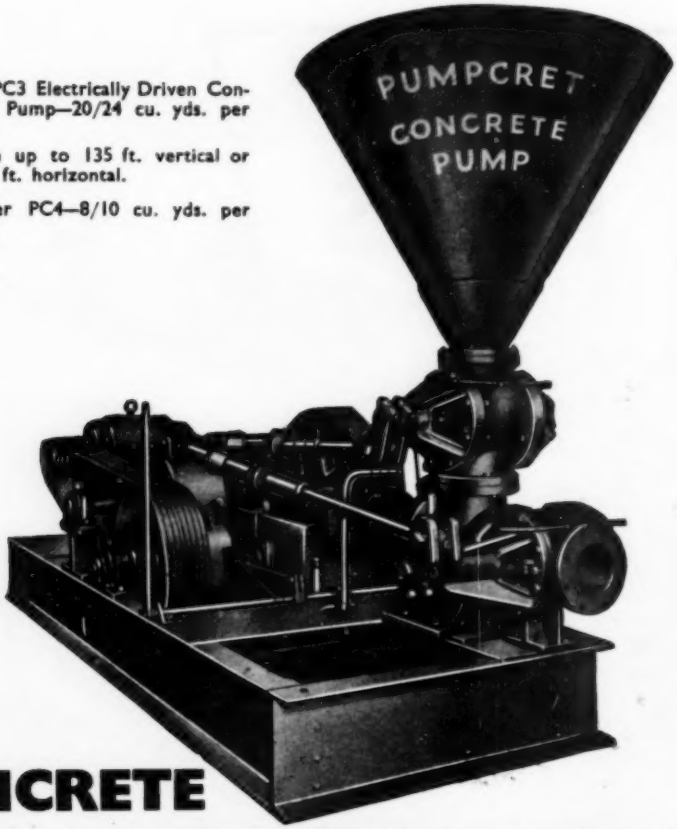

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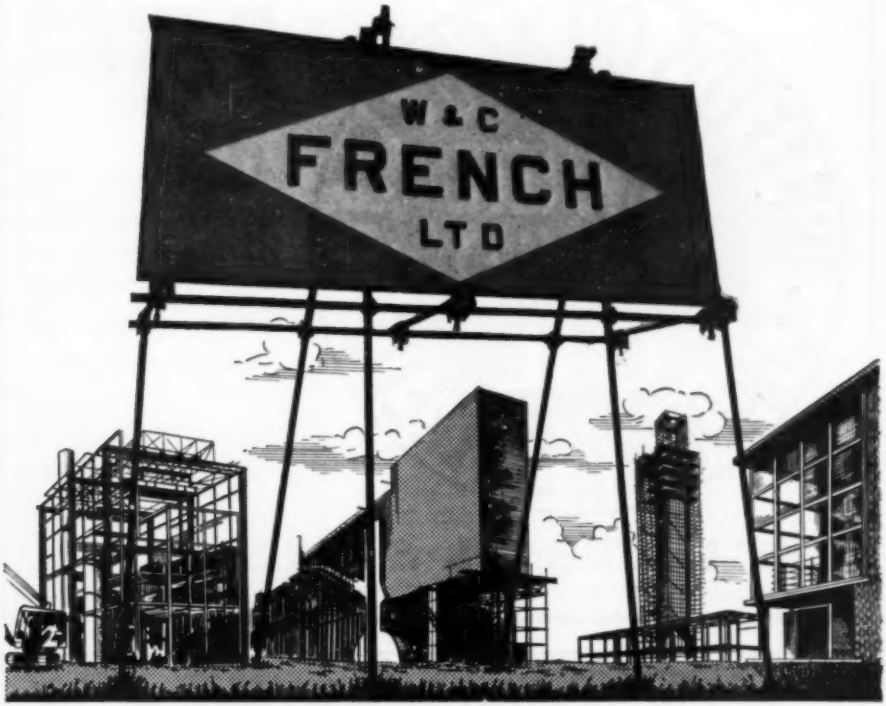

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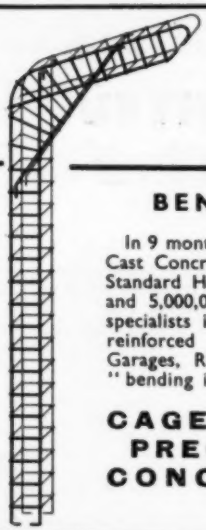

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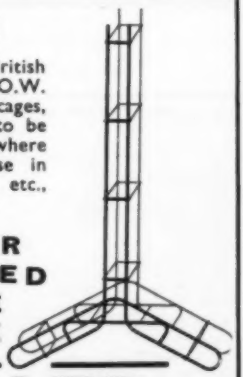

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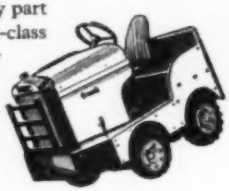

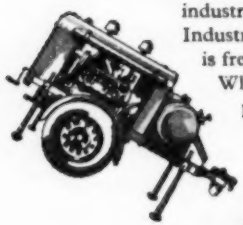

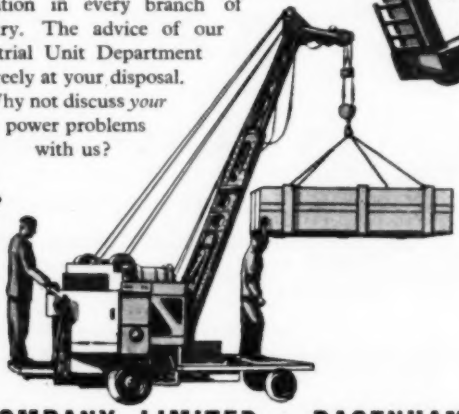

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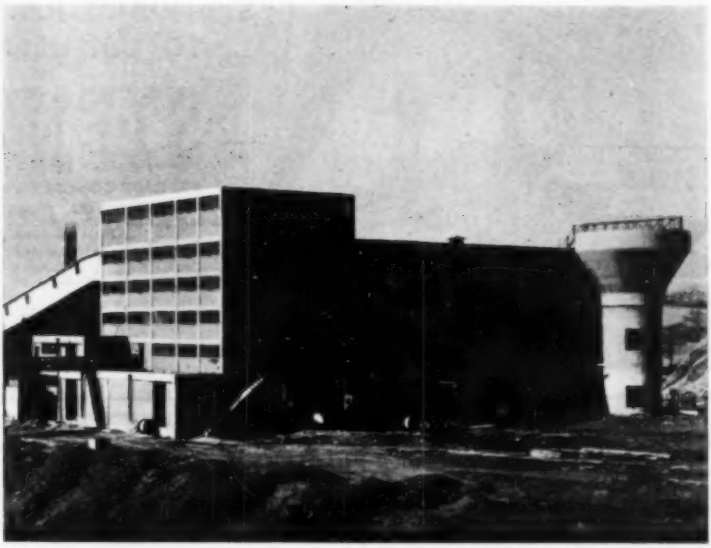

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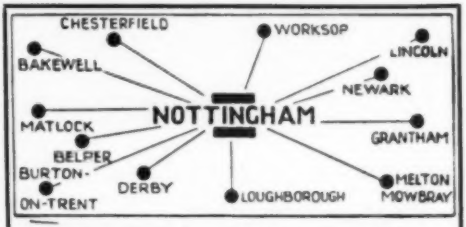

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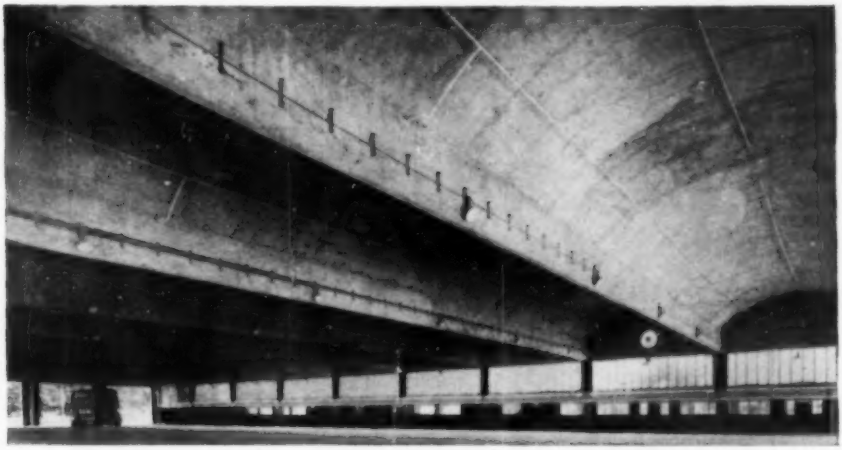

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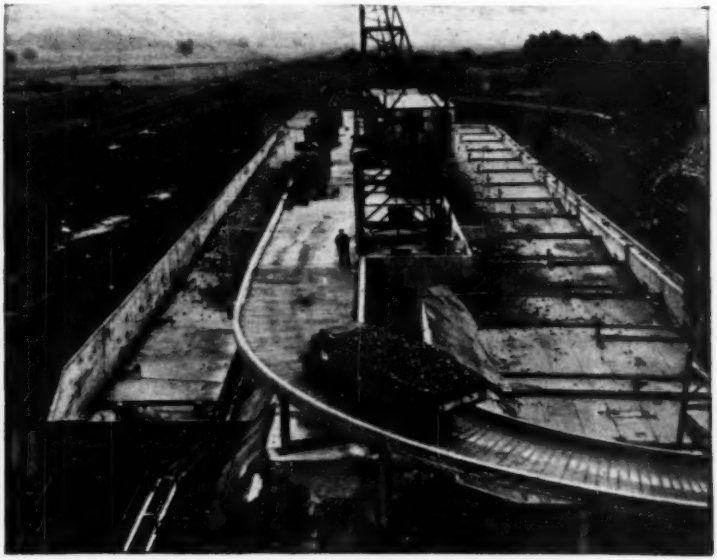

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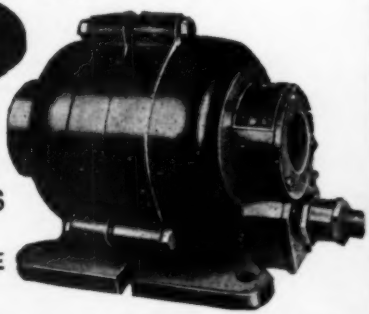

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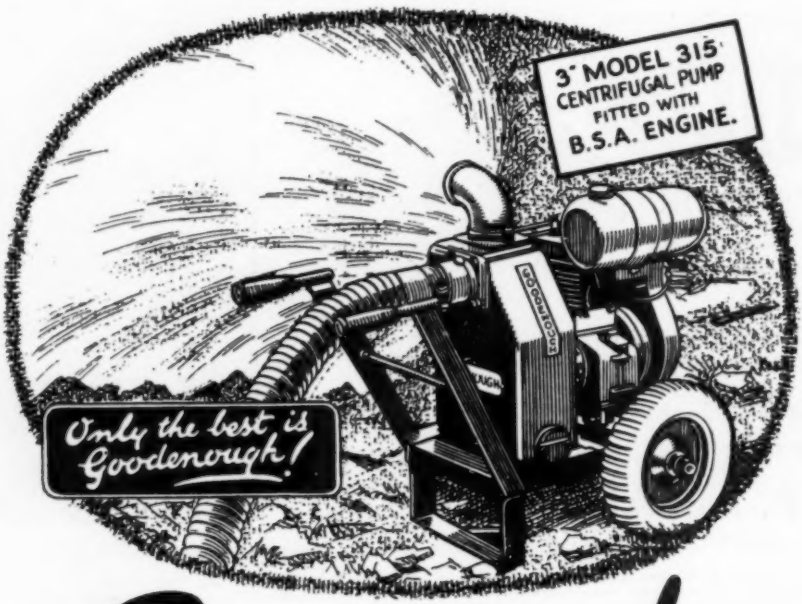

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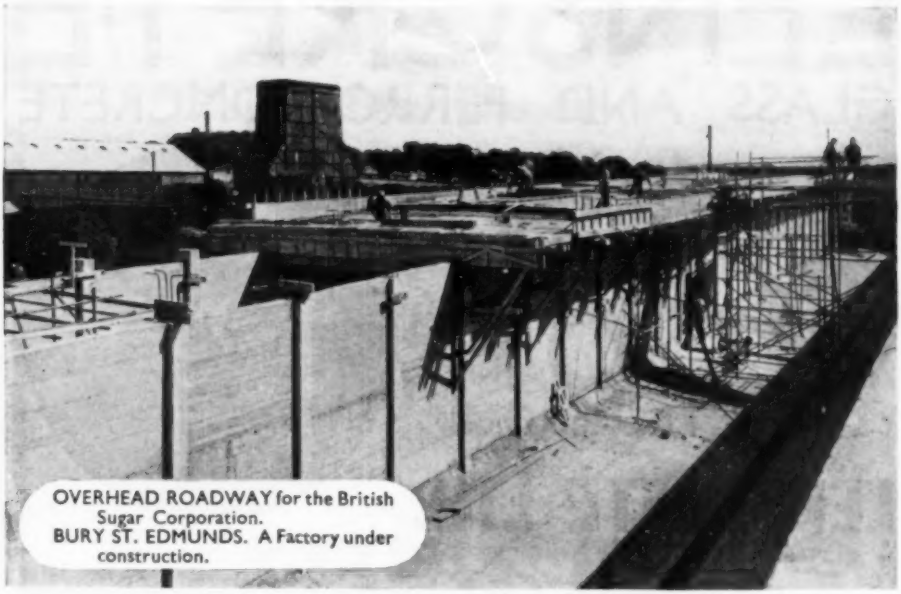

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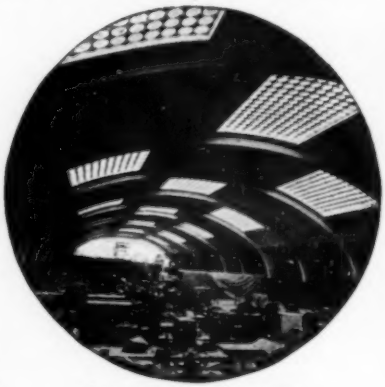

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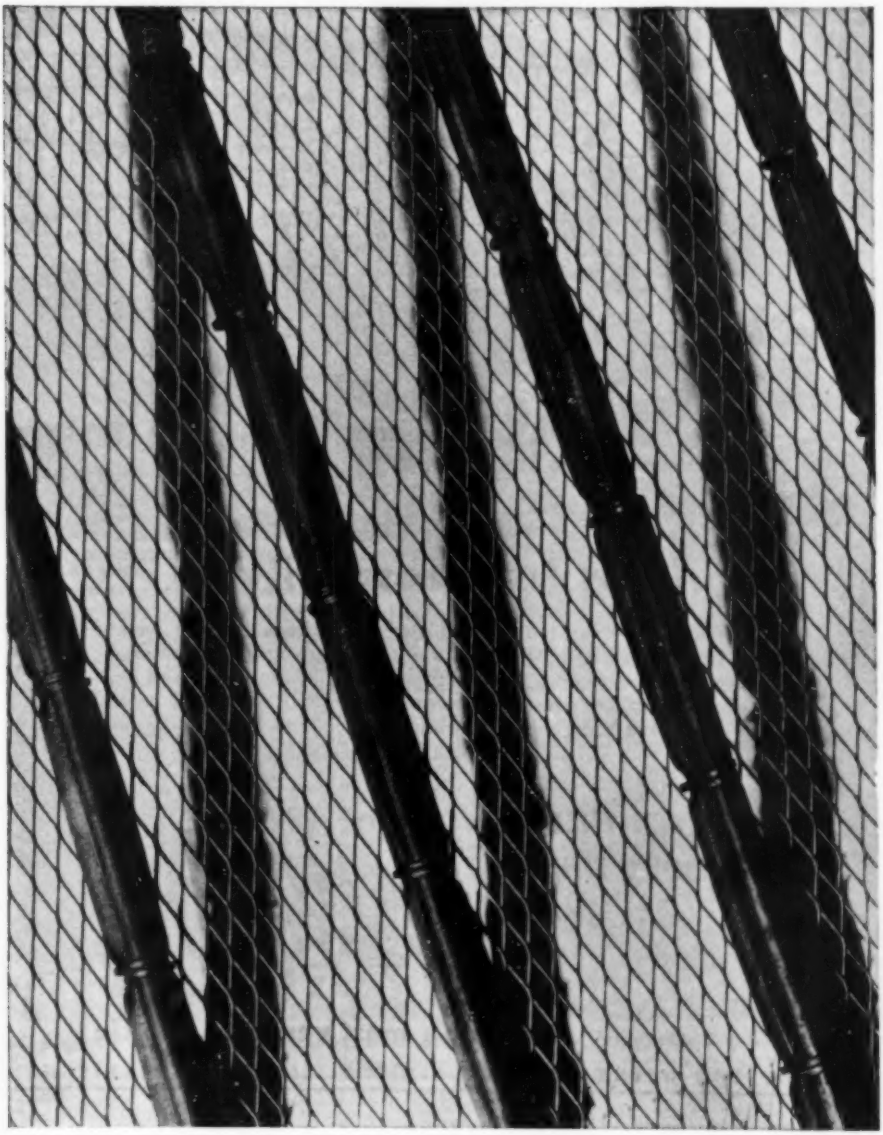

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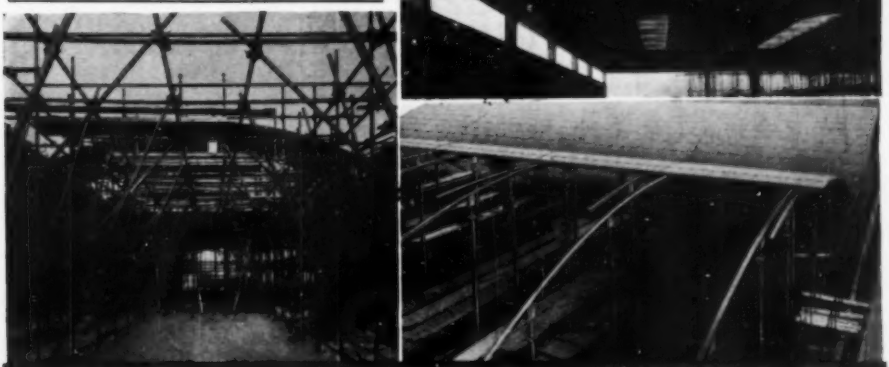

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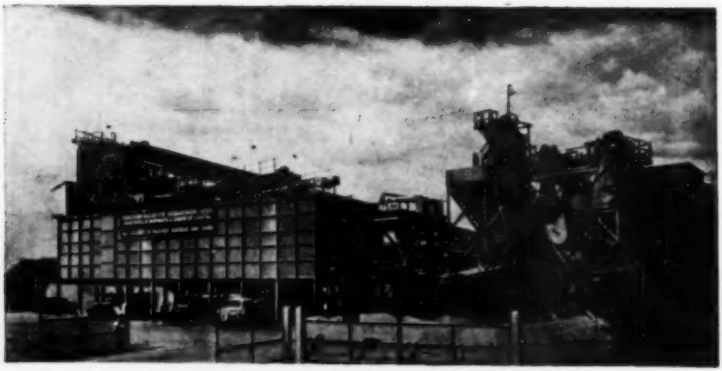

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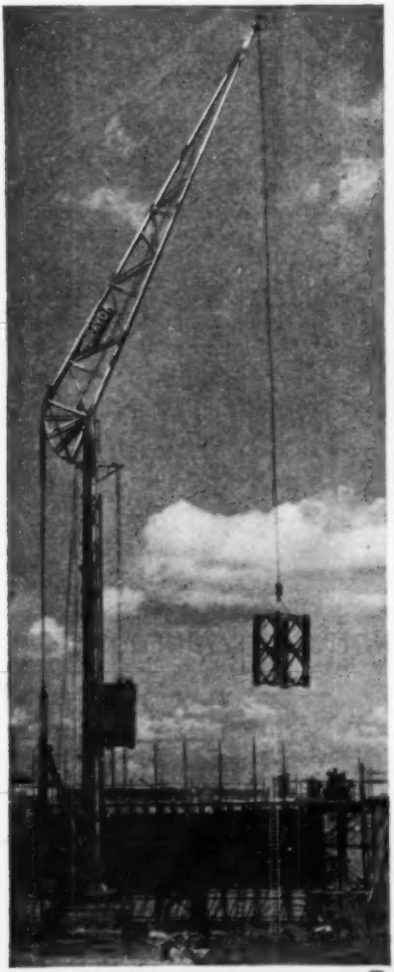

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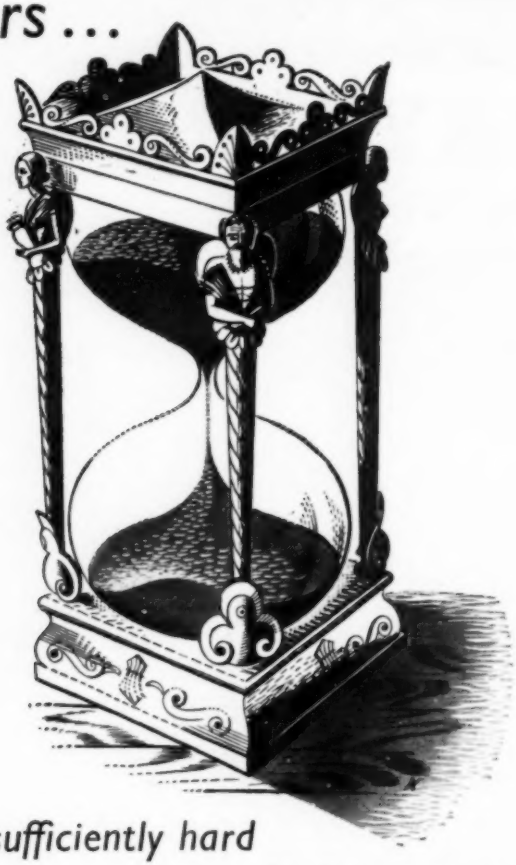

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Volume XLVII. No. 9. LONDON, SEPTEMBER, 1952

EDITORIAL NOTES

Higher Stresses in Mild Steel Reinforcement.

WITH a view to economising in materials, the Director-General of Works of the Ministry of Works, the Civil Engineer-in-Chief of the Admiralty, the Director-General of Works of the Air Ministry, and the Director of Fortifications and Works of the War Office have prepared a report entitled "Economy of Building Materials" * in which recommendations are given on saving materials and the use of substitute materials in building and civil engineering works. In a foreword it is stated that these recommendations have already been adopted by the Ministry and the Services, and the hope is expressed that they will be adopted for all constructional work. The report emphasises the saving of steel that is possible by the use of reinforced concrete, and particularly prestressed concrete, and urges that these materials be used wherever possible in place of structural steel; it is to be hoped that ample supplies of reinforcement bars and prestressing wire will be made available so that this saving will be possible.

The report states that the Ministry of Works and the Services have now adopted a working stress of 20,000 lb. per square inch in mild steel reinforcement bars. It is only four years since the Codes of Practice Committee (now the Council for Codes of Practice), which is under the ægis of the Ministry of Works, was responsible for the publication of British Standard Code of Practice No. 114, which states that 18,000 lb. per square inch is the maximum permissible working stress in mild steel reinforcement bars. Since that date there has been no improvement in the quality of mild steel or cement or of concrete in high-class work, so that it can hardly be said that this decision is based on improvements in the quality of materials or in methods of making concrete. The committee responsible for B.S. Code No. 114 is this month to consider the revision of this document, but it seems that its labours may have been anticipated so far as concerns the working stress in mild steel bars. There is no doubt that in high-class work a stress of 20,000 lb. per square inch could be adopted and still leave an ample margin of safety. A committee responsible for a code of practice, however, has to bear in mind that the code will apply to all structures with varying qualities of materials, supervision, and labour, and not only to high-class work. It may indeed be pointed out that when inferior aggregates, supervision, and labour only are available it is not always easy to obtain the crushing strengths

* H.M. Stationery Office. Price 2s.

required by the B.S. Code without increasing the cement content. The report places more importance on saving steel than on saving cement, no doubt because steel is more urgently needed for the equipment of the Services, for both steel and cement could be exported in greater quantities if they were available.

This recommendation is likely to cause uncertainty in the minds of those responsible for the design of reinforced concrete. The B.S. Code was produced by a committee appointed by the Institution of Structural Engineers at the request of the Codes of Practice Committee. The draft of this code was submitted to all organisations intimately concerned with reinforced concrete, and their views were taken into consideration. The code is not mandatory, but is issued for guidance only. The committee responsible for the new recommendations comprised only the Works Directorates of the three Services and the Ministry of Works. Its recommendations are mandatory for structures built for the Ministry and the Services, but for other structures they are not obligatory. The foreword to the report, however, states that " the co-operation of the architectural and engineering institutions is of the greatest importance to ensure that the findings of the report are not only given the widest possible circulation but are everywhere understood and applied." It would therefore appear that the recommendations of this report are intended to have precedence over the recommendation of B.S. Code No. 114 that the working stress in mild steel reinforcement bars be 18,000 lb. per square inch. Whereas the B.S. Code may be assumed to represent the consensus of opinion of the profession, the new recommendations seem to be based only on the views of a few people within the Ministry and the Services, and may therefore be an expedient to save steel by lowering the factor of safety considered to be necessary as recently as four years ago. Engineers will, as in the past, use their own judgment on what working stresses they will adopt in structures other than those for the departments which require that the higher working stress in steel be adopted. It is, however, a complication that designers should be advised to use different stresses by different committees. It is also recommended that 30,000 lb. per square inch be adopted for high-tensile steel.

So far as saving cement is concerned, the report suggests that greater use be made of plain concrete whenever possible. Plain concrete could certainly be used more frequently in foundations, but for retaining walls the thickness of plain concrete walls would often be so great that they would not always be practicable. In such cases the report suggests that the engineer must use his discretion whether to use reinforced or plain concrete. A greater use of soil-cement mixtures is suggested for roads where the subgrade is suitable and the anticipated traffic light ; where concrete roads are necessary the report suggests that the subgrade be consolidated sufficiently to allow the slab to be as thin as possible.

Other recommendations are that greater use be made of slabs, of hollow-tile construction, or of trough construction for spans between 15 ft. and 35 ft. This would reduce the story height of buildings by eliminating beams and thereby effect a small economy in the height of walls and their finishes ; in addition, the omission of internal columns and the simplicity of construction may well show a saving in cost. The report emphasises the advantage of careful analysis of the stresses in structures and of careful consideration of the loads likely to be imposed upon them, for careful design and detailing can effect savings in materials and in cost whatever the material or the working stresses adopted.

Centering for Shell Roofs.

THE following is an abstract of a paper by Mr. H. F. Rosevear read at the recent symposium on shell roof construction arranged by the Cement and Concrete Association.

The roof for which the centering was designed covers an area of 151 ft. 6 in. by 400 ft. 6 in., and the barrels are in three bays spanning the shorter dimension. The height to the top of the supporting columns is 13 ft., and the barrels have a rise of 5 ft. The columns, gable ends, and the stiffening beams between the internal columns were precast on the floor of the building, and erected by a mobile crane. The columns weighed 1 ton each, and the stiffening members weighed from 3¾ tons to 4½ tons each. These were erected in advance of the construction of the barrels.

Two transverse expansion joints are provided across the width of the building, and at these joints the central bay had upstanding beams in the valley so that the barrels on each side of the expansion joints would not need propping when the centering was struck, and so enable the centering for the barrel to travel under the valley in the longitudinal direction. This arrangement reduced the stiffening requirements for the passage of the centering, but meant that all centering moving in a longitudinal direction must pass under the expansion joint in the central bay. The end barrels on each side of the expansion joints were not provided with vertical stiffening beams at the valley, so that the propping had to remain under these valleys until the abutting barrel was concreted and had matured.

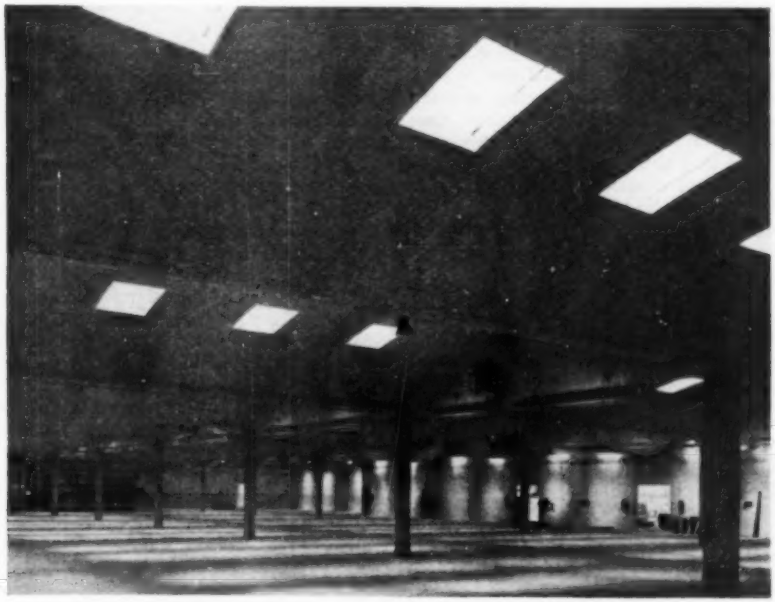


Fig. 1.—Interior View.

Fig. 2 is a plan of the structure, and the numbers in the bays indicate the sequence of concreting the barrels. The arrangement of columns and stiffening diaphrams formed 45 separate vaults, the size of each being about 50 ft. by 26 ft. 5 in. with a radius of 20 ft. Five barrels were concreted at a time, so that a

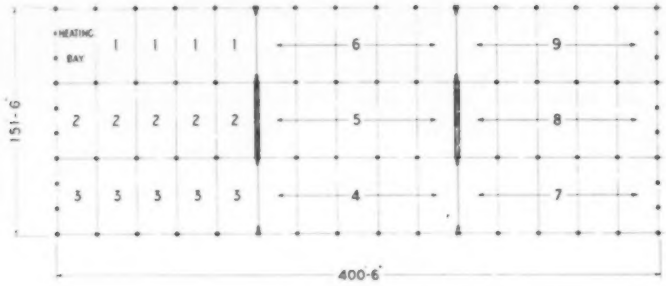


Fig. 2.

set of five centres was used nine times. It was necessary to pass all the centres through the central bay when moving them in a longitudinal direction, so that some of the centres changed direction twice. *Fig.* 3 shows the precast columns and stiffening diaphragms and some of the centres for the barrels in position ; in the foreground is seen a centre ready for lifting.

The centres were the full area of a barrel and light enough to be lifted and moved. The design of the centering and supports, which was to some extent restricted by the scarcity of some materials, is as follows. The soffits of the barrels were to be lined with ½-in. ceiling board, and this was supported by 20-gauge steel sheet stiffened with V-shaped longitudinal ribs 1 $\frac{13}{16}$ in. deep of the same material and welded to the underside at about 8-in. centres. This shutter (which is patented) is capable of supporting the concrete in the barrel over a width of 2 ft. 11½ in. Every 2 ft. 11½ in. a transverse tee-shaped curved rib supports the longitudinal ribs. This curved rib is of 2-in. by 2-in. by ¼-in. mild steel rolled to the curve with the flange downwards. Small fishplates are used to join the transverse ribs where necessary. Holes were provided in the flanges of the transverse ribs so that they could be wired to the top of the supporting framework over

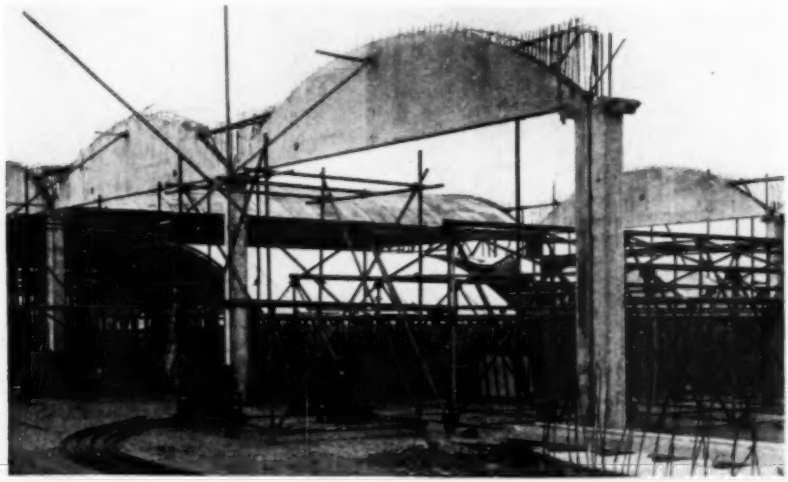


Fig. 3.—Showing Precast Columns and Diaphragms. Centering for One Barrel in Position and another Centre being Raised.

which they passed. The depth of the longitudinal ribs allowed the edge of the steel sheeting to be passed over the top edge of the transverse rib, so forming a flush curved surface to receive the ceiling board. Holes were provided through the longitudinal ribs at the ends so that they could be wired to the transverse ribs. This construction is shown in *Fig.* 4.

The framework supporting the centering was a system of Warren girders with joints 5 ft. 11 in. apart. This arrangement gave the least number of joints consistent with low first cost and lightness. Due to the scarcity of small structural steel sections these girders were made of standard steel scaffold tubes. *Fig.* 5 shows a detail of the framework. The maximum available depth for the girder was used having in mind that the shutter had to be lowered on to 2-ft. gauge bogies for travelling and passing under the valleys and barrel stiffeners.

With one scaffold tube to form the bottom boom of the girder, four longitudinal girders were required to carry the centering, and scaffold tubes were used to form transverse Warren girders at the ends to enable the whole to be carried at the corners. The bottom booms were kept at one level and the top booms varied to suit their position on the curve of the barrel. The top boom of the longitudinal girders was 2½-in. bore 5-gauge tube, as this member carried a central concentrated load from the curved ribs supporting the sheeting and the wet concrete. The stress in the tubes from the primary forces of the girders was about 7 tons per square inch so it was necessary to ensure that members in all directions truly intersected at the nodal points and that there were no eccentricities that would cause secondary stresses at the connections.

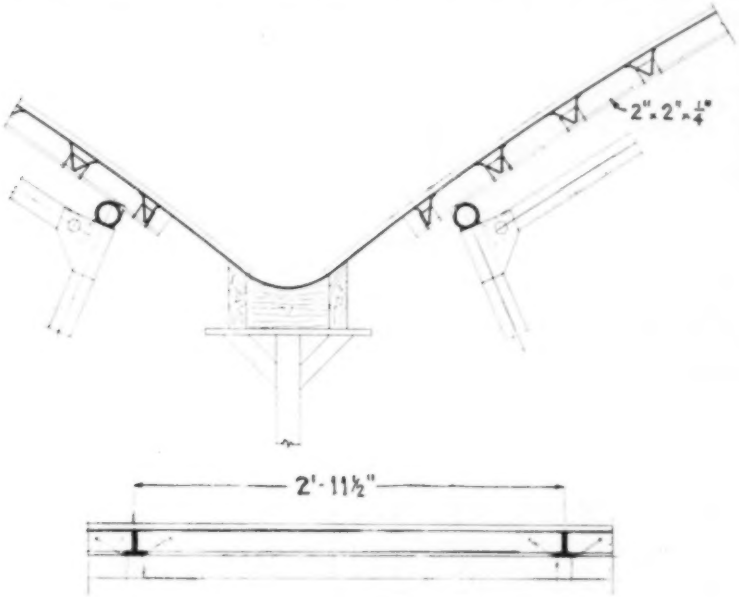


Fig. 4.

The longitudinal girders were made in three pieces with welded intersections and were assembled and joined together on the flat. They were then turned to the vertical and the transverse and bracing members fixed by bolts passing through holes in gusset-plates previously welded to the longitudinal girders. Diagonal bracing was then added at the top and bottom boom levels in order to restrict racking stresses during lifting and travelling ; this was confined to the central bays

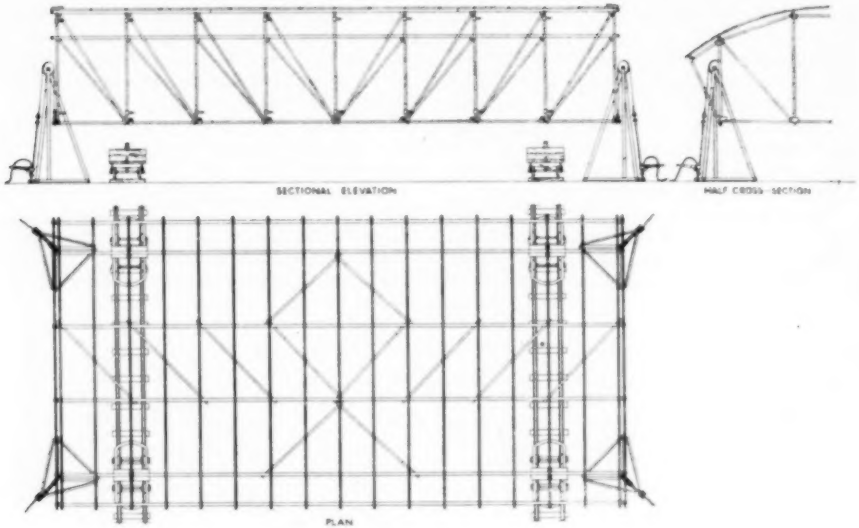


Fig. 5.

in each direction, using ordinary scaffold-tube couplers for connections. This bracing kept the frame square in plan but allowed reasonable flexibility in the vertical direction to allow for the variation in level at the corners which would occur during raising and lowering and setting to the exact level required. The lower booms at the extreme bottom corners were extended and provided with stiffening to

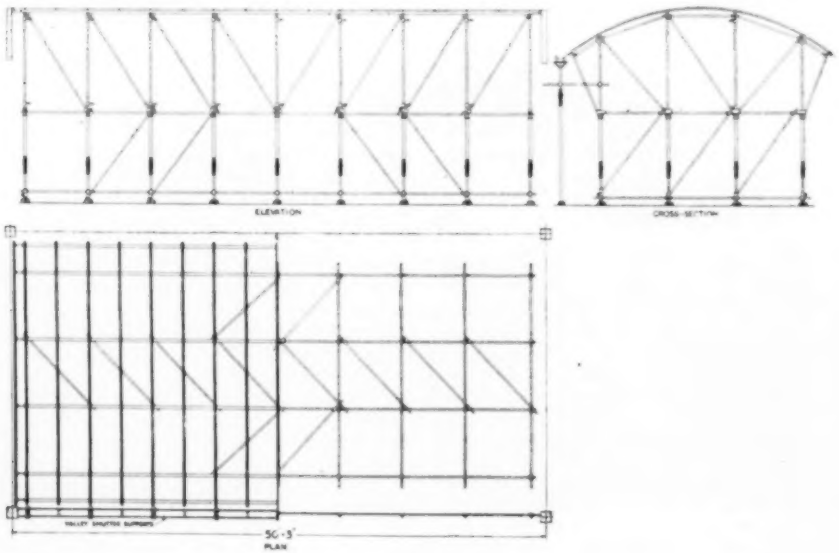
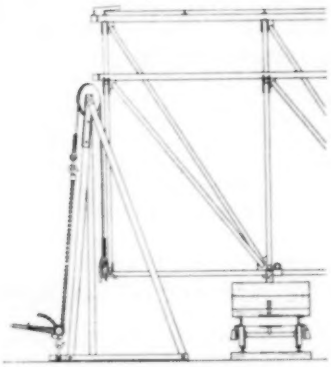
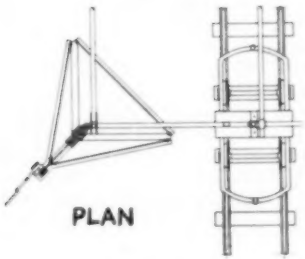


Fig. 6.



ELEVATION



PLAN

Fig. 7.

carry the shackle on the lifting tripods for lowering and raising.

The longitudinal girders had to be shorter than the distance between the faces of the columns and the diaphragms between the columns, and in the ends of the top booms short lengths of smaller tube were placed which could be moved in and out. These sliding tubes were pulled out to touch the barrel diaphragms to carry the end transverse ribs and sheeting, and were pushed in during the lowering of the centering in order to pass between the columns.

When the centering was lowered a few inches the sheeting at the ends was pushed up on to the main sheeting a little and the end transverse ribs and sliding tubes were moved back to give the required clearance for travelling. The spaces between the edges of the centres at the valleys were covered with steel sheets shaped to the soffit of the valley and fixed to longitudinal timbers fastened together and supported on adjustable props. Horizontal scaffold tubes were added between these adjustable props and those under the main centering when they had been placed for concreting.

As the girder would carry only its own weight and that of the sheeting, saddle-pieces were fixed under each panel point, under which adjustable props were screwed up so that each prop carried the weight of the centre and of the concrete transmitted downwardly through any vertical member of the truss. The vertical members therefore functioned within the girder and were strong enough as struts to transmit to the base slab all the weight imposed upon them. The props were braced horizontally and diagonally. *Fig.* 6 shows the shuttering with props and bracing in position, and the valley shutter and support in position ready to receive concrete.

The centering was raised and lowered with one set of four tripods which could be easily moved about and applied at the corners of any of the centres. The tripods (*Fig.* 7) were also used for lifting centres in the central bay when it was necessary to turn the bogies through 90 deg. The tripod was made of 3-in. diameter mild steel tubes with a pulley at the top. The lower ends were retained in position by mild-steel angles, which were brought together and outward from one leg of the tripod to provide anchorage for a 1½-tons pull with a chain 7 ft. long. A wire bond from the lever was continued upwards over the top pulley and down to a shackle which could be fixed to the projections provided at the extreme bottom corners of the centres. By removing the extreme corner adjustable props first the tripods could be placed in position and the lever operated so that all the other props could be removed, leaving the centre suspended only at the tripods. The tripod could be positioned only one way at the corners of the centre, and when the load was lifted it was in the centre of the base of the tripod, thus providing maximum stability.

The lifting and lowering of the centering was easy, with a ganger shouting to keep the men in time to ensure that the centre was kept horizontal. When the centre was moved it was lowered on to 2-ft. gauge bogies on rail tracks. The

bogies were provided with a cross-bearer and received the saddle-bearer fixed to the bottom booms of the outer longitudinal girders at the first internal panel points. The saddles on the cross-bearers on the bogies could be set in any direction, and the tracks with the bogies ready to receive the centre were laid in the direction of travel required. To change the direction of travel the tripods were placed at each corner again, the centre lifted a few inches, and the track and bogies turned to the new direction. It was necessary to change direction twice every time a centre was moved in a longitudinal direction in the central bay under the valley beams.

The first concrete was placed for the barrels on 23 January, 1951, and completed on 3 June of the same year, giving a progress rate of three days for each barrel.

The Effect on a Shell Roof of the Settlement of Supports.

In an article in "Travaux" for July, 1952, M. J. Larras gives a report of tests carried out to determine the effects of vertical movement of the supports of a shell roof. The shell, which had no cross ties other than the solid gable beams nor any longitudinal edge beams, was of elliptical cross section. The major axis was 41 ft. 9 in., the minor axis 9 ft. 2 in., the span 83 ft. 9 in., and the thickness of the slab $2\frac{3}{4}$ in. The tests consisted in measuring deformations of the concrete and of the reinforcement at various points on the surface of the shell and the vertical load at one corner of the shell which was raised relatively to the other corners by means of jacks. This corner was raised five times through a vertical distance of 10 in. and ten times through 6 in. The tests were carried out over a period of months.

The conclusions drawn from the results were that cylindical shells of this form will safely withstand large vertical movements of one support relative to the others so long as they are freely supported. Although the usual theory for computing torsional stresses in thin open sections gives an idea of the maximum stresses in the concrete there may be considerable inaccuracy in the values so calculated due to the unpredictable effects of the stiffness of the supporting columns and the consequent resistance to free movement of the shell. Tests on models showed that suitable coefficients may be obtained for each particular case which allow these latter stresses to be calculated with sufficient accuracy. It is emphasised that the limited movements considered did not bring into action the full resistance of the reinforcement and that the conclusions are applicable only to geometrically similar shells. Further measurements of strain on many other structures of different shapes and dimensions are necessary to permit generalizations to be made.

Methods of Testing Concrete.

The British Standards Institution has issued in one booklet (price 10s.) a number of standard methods of testing concrete, some of which appear in other specifications. The tests included are: Sampling fresh concrete on the site; Slump test; Compacting factor test; Analysis of freshly-mixed concrete; Determination of weight per cubic foot of freshly-mixed concrete; Making and curing compression test cubes in the laboratory; Making and curing compression test cubes on the site; Test for compressive strength of moulded cubes; Determination of the compressive strength of concrete using portions of beams broken in flexure ("equivalent cube" method); Making and curing flexural test specimens in the laboratory; Test for flexural strength of moulded flexural-test specimens; Making and curing test specimens for determination of the modulus of elasticity by means of an extensometer; Determination of the modulus of elasticity by means of an extensometer; Determination of the modulus of elasticity by an electrodynamic method; Drilling, examination, measurement, and testing of cores from concrete; and Determination of changes in length on drying and wetting (initial drying shrinkage, drying shrinkage, moisture movement).

Shell Roofs at an Omnibus Garage.

THE main structures of a new garage at Peckham (Bull Yard, London, S.E.) for the London Transport Executive consist of a two-story administration building and the shell roofs to the garage area and repair bays. In addition there are a boiler house, an underground storage-pit for oil-fuel, and various single-story annexes on two sides. A photograph of a model of the structure is given in *Fig.* 1, and *Fig.* 2 is an interior view of the completed building. The roof over the garage has an area of about 53,000 sq. ft., comprising twelve bays each 110 ft. long and 40 ft. wide. Natural light in the garage area is provided by side-lights in the shells; the area of lighting openings is about 15 per cent. of the floor. For the repair bays, where better lighting is necessary, north-light shell construction gives a lighting area of 33 per cent. of the floor.

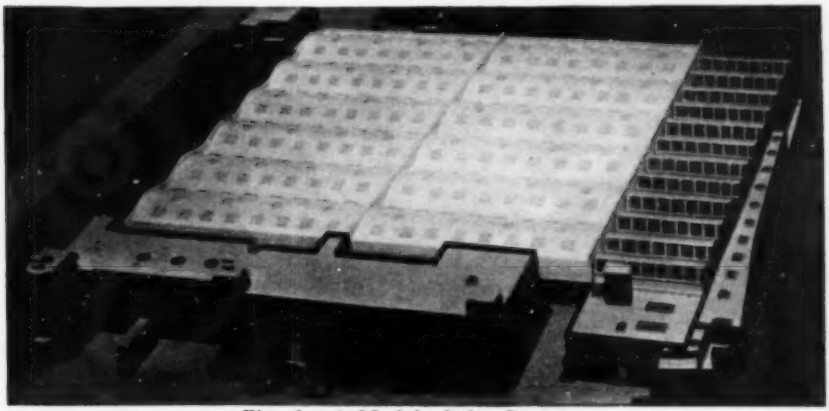


Fig. 1.—A Model of the Garage.

Holes dug to a depth of 8 ft. revealed 3 ft. to 4 ft. of made-up ground overlying sandy brown clay of poor bearing quality. Boreholes were sunk to a depth of 40 ft., and at a depth of 34 ft. firm red marl was encountered. The main structures are carried on bored piles (of which 250 have been formed) continued down to the red marl and capable of carrying a working load of 50 tons each. Loading tests on a single pile and a pair of piles resulted in a final settlement of 0·119 in. and 0·025 in. respectively after being loaded for one week with 75 tons per pile.

The columns carrying the roof are supported on beams around the perimeter of the building and also on two internal columns of 32 in. diameter spaced 80 ft. apart. The internal columns carry a load of 565 tons each and support a central beam (*Fig.* 3) carrying the interior ends of the shells. The shells (*Fig.* 4) are generally 3 in. thick, increased to 4 in. at the ends and also thickened at the junctions with the valley beams. The lighting openings are 6 ft. square with corners rounded to 9-in. radius, eight lights being arranged along each side of each shell; in addition there are four circular openings 4 ft. 6 in. diameter in the crown which carry sheet-metal louvred ventilators capped with a domed light (*Fig.* 5).

The central beam is 12 ft. 6 in. deep and 2 ft. 6 in. wide and is divided vertically by an expansion joint 1 in. thick to allow the roof some freedom of movement due to thermal changes, to avoid continuity between adjacent shells, and to simplify construction. The twenty bays of north-light roofs (*Fig.* 6) over the repair bays are generally 3 in. thick, 20 ft. wide and span 50 ft., and cover a floor area of 12,000 sq. ft. Additional columns between the garage and repair bays divide the total width of the building to suit the

arrangement of repair pits and to support large folding doors.

The roofs over the garage and their supporting columns are separated from the administration building by a 2-in. gap, each structure being supported independently. To prevent movement of the shell being transmitted to the brick panel-walls, open joints are provided between the sides and backs of columns and the brickwork. The walls are carried on

the need for rubbing down the finished soffits, which were to be colour-washed, hardboard was laid over the sheeting. The surface obtained was satisfactory for colour treatment, and five uses were obtained from the hardboard. The square roof-lights were large enough for all the components of the centering to be passed through and re-assembled in adjoining bays.

As speed of construction was important,

Fig. 2.—Interior View.

beams at ground-level spanning between the columns, with a soft joint between the tops of the brick walls and the soffits of the beams above. At the entrance to the garage a ledged edge-beam with a sheet-lead seating provides a sliding joint for the roof between the garage and the part of the structure over the entrance.

The shuttering for the roofs (*Fig.* 7) comprised light timber trusses spaced 7 ft. apart and supported on tubular scaffolding. The sheeting consisted of timber panels 7 ft. long (which had been used in the construction of caissons for Mulberry Harbour) laid across the trusses. To reduce

it was considered an advantage to concrete all the tie-beams and valley beams up to springing level prior to casting the curved slabs. Construction joints were arranged at the midspan of the ties and beams to form tee-sections (on plan), and each tee was cast in one operation. One-half of the garage roof beams was cast in this manner before concreting of the shells was commenced, and the shuttering to the beams sides was removed as early as possible for re-use. The beams for the second half of the roof were concreted in a similar manner while the slabs on the first half were being concreted. The con-

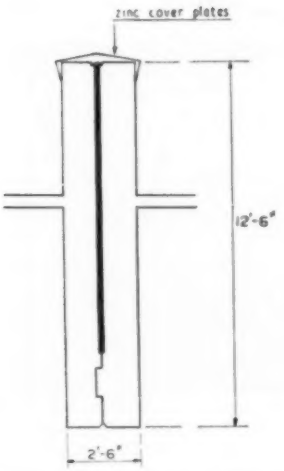


Fig. 3.—Cross Section of Central Beam.

struction joints in the roof slabs are along the crowns and valleys longitudinally and at mid-span transversely; concreting commenced at the valley and worked up to the crown. *Fig.* 8 shows the roof during construction.

No difficulties were found in compacting concrete with a slump of about 3 in.; any cracks in the top surface due to shrinkage, etc., were grouted while the concrete was still green. Sprinklers were provided for curing the concrete during the summer, but were not used as the rainfall was sufficient for the purpose. The shuttering of the roof slabs was struck three or four days after concreting, and the beam-sides about two days after completion. The soffits of the beam-boxes were left as long as was conveniently possible, but the shortest time of striking was about ten days from the completion of a bay. Due to the large number of roof openings the steel fixing was more complicated than is usual in this type of roof,

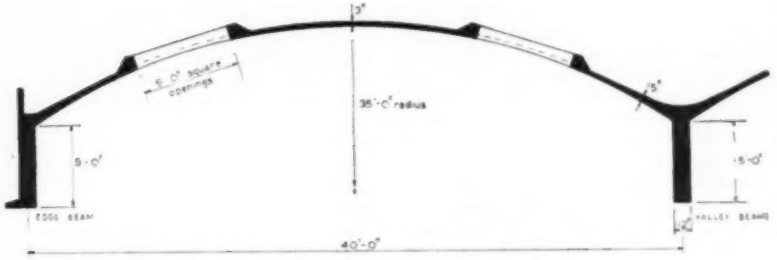


Fig. 4.—Typical Section of Shell Roof.

Fig. 5.—Lighting and Ventilating Openings in Roof.

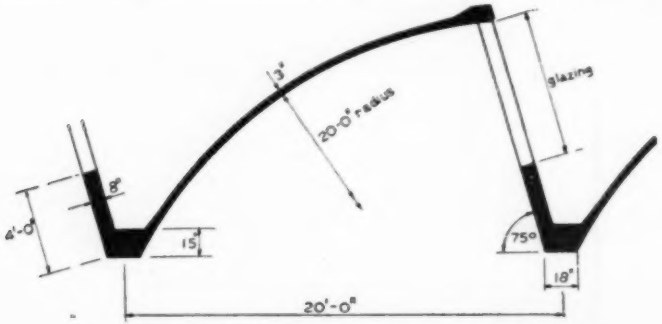


Fig. 6.—Typical Cross Section of North-Light Roof.

but was completed in two days for each bay. One complete bay of 110 ft. by 40 ft. was erected in one week.

The method of constructing the north-light shell roofs was similar to that employed for the barrels. These slabs were concreted without top shutters except for the first 2 ft. from the valley, where top shutters were wired to the reinforcement, with spacer-blocks to ensure the correct spacing and cover for the bars. The roof was covered with 1½ in. screed of vermiculite (graded in the valleys for drainage) to provide insulation. The roof was finished with two layers of bituminous roof-felt sealed down with hot bitumen. The roof is drained from the central beam to the east and west boundaries, where purpose-made sheet-metal hoppers built into the valleys discharge into rainwater pipes fixed to the columns on the inside of the building. The top edge of the 1-in. wide expansion joint in the central beam is protected from the weather by a capping of sheet zinc clipped over the projecting top of the beam (*Fig.* 3).

The administration building is of normal reinforced concrete construction and includes offices and medical treatment rooms over the 70-ft. wide main entrance of the garage. The floor and roof of the medical rooms are carried by two portal frames, the beams forming the breast wall at floor level. In the office block the columns are in the walls and partitions to allow as much unobstructed area as possible in the hall at ground level and the canteen and kitchen on the first floor. A storage basement is provided under the hall; sulphate-resisting Portland cement was used in the construction of the basement on account of a deposit of salt found in the ground, and similar cement was used for the piles. The basement is not on piles, and is completely free from the general structure. The roof of the administration building is insulated with cork slabs 1 in. thick and, like the garage roof, is finished with two layers of bituminous roofing felt. The boiler house is partly underground with a fuel-hopper at ground level. Oil-fuel for the vehicles is stored

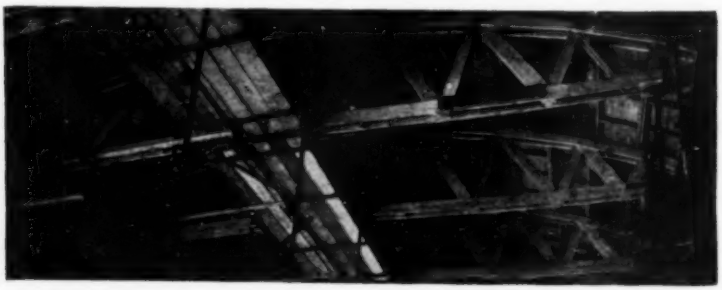


Fig. 7.—Centering for Shell Roofs.

Fig. 8.—Roof of Garage during Construction.

in steel tanks with a capacity of about 30,000 gallons accommodated in a pit about 20 ft. deep into the ground.

The architects are Messrs. Wallis Gilbert & Partners in association with Mr. Thomas R. Bilbow, F.R.I.B.A., Chief Architect of the London Transport Executive. Messrs. John Liversedge & Associates were the consulting structural engineers; this firm's Arch-span construction was used for the roof of the main garage and their North-span construction for the roof of the repair shops. Mr. A. Sackey, M.I.Struct.E., was responsible for the structural design. The main building contract was carried out by Messrs. J. Jarvis & Sons, Ltd., and the reinforced concrete work in the main buildings by Messrs. Richard Costain, Ltd.

Combined Timber and Concrete Piles.

It is necessary for their preservation that timber piles be kept at a constant moisture content, and in Holland they are frequently driven below low water level with in-situ concrete extensions above them.

A method which may be used where extensions up to 5 ft. long are required is due to Mr. F. Fennema, of Hardinxveld, Holland. A watertight bag of bituminized paper is made, strengthened by wires circumferentially and by wooden ribs vertically, and this is placed on the timber pile after the latter has been driven to its final set. A cage of reinforcement is placed in the bag and concrete poured through a tremie, the paper serving to form shuttering and to protect the newly-placed concrete from damage by ground water. After placing the concrete the space around the paper mould is filled with earth. Two types of bag are used. The first, for use where only vertical pressure occurs, has a rebated seating 2½ in. deep on the timber pile and is without circumferential reinforcement; it may be strengthened by driving a ragbolt in the centre of the timber pile. The second type has an 8-in. deep rebated seating and has helical reinforcement. The bags are made in six lengths with a diameter at the top of 11 in. for the first type and 12½ in. for the second type.

It is claimed that this method is cheaper than the method of driving concrete piles on top of wooden piles described on page 220 of the July, 1952, number of this journal.

Book Reviews.

"Examples of the Design of Reinforced Concrete Buildings." By C. E. Reynolds. (London: Concrete Publications, Ltd. 1952. Price 10s.)

THIS book is the successor of the same author's well known work "Practical Examples of Reinforced Concrete Design." The present book has been brought up to date and gives examples of the calculations and drawings required for a typical multiple-story building based upon British Standard Code of Practice No. 114 and its associated sub-codes and is also in accordance with the new recommendations for loads given in the revised Code of Practice No. 3, Chapter V, "Loading," which was issued in August, 1952, and is reviewed on page 279 of this number. In addition to alternative examples of the design of the principal parts of a building as a beam-and-slab structure and as a flat-slab structure, the author comments upon and gives details of the application of the codes of practice in general. The book contains tables summarizing the requirements of the codes, tables and graphs for the rapid assessment of bending moments on continuous beams, the shearing resistance of groups of bars, the allowable loads on both concentrically and eccentrically loaded columns, and many other tables and graphs which reduce labour in the design of most classes of reinforced concrete structures.

"Studies in Cement-Aggregate Reaction." Parts IX–XV. (Melbourne: Commonwealth Scientific and Industrial Research Organisation. No price stated.)

PAPERS by Mr. H. E. Vivian and Messrs. R. H. Jones and H. E. Vivian, describing the results of investigations on the effect on concrete of aggregate containing alkali and other impurities and on the effect of the water content on the expansion of mortar.

"Aglomerados de Hormigon Hidráulico." By Patricio Palomar. (Barcelona: Editorial Bosch. Price 170 pesetas.)

IN recent years many noteworthy reinforced concrete structures have been built in Spain. These have been described in the Spanish technical press. In this comprehensive work of more than 500 pages the author deals with the precast concrete industry. The uses and methods of manufacture of all classes of precast concrete products are dealt with in detail, and some account is given of the manufacture of prestressed precast concrete products and of the uses of asbestos-cement products. In addition to the methods of Spanish manufacturers, descriptions are given of the methods used in other countries. The book is well illustrated with photographs and drawings of moulds and products.

"Forces in Framed Structures." By T. Lyle Morgan. (London: E. & F. N. Spon, Ltd. 1952. Price 25s.)

THIS book describes and gives examples of the usual methods of numerical and graphical analysis of the forces in and the deflections of pin-jointed plane frames and simple space frames. The author emphasizes the method of equilibrium of joints which, as he says, has the advantage of simplicity. For calculating deflections of trusses the method described is that of virtual work. It is strange that the Williot-Mohr method is so frequently ignored, as when the method of virtual work is used to calculate deflections of trusses only one component of the deflection of a joint can be calculated at a time. To obtain both the horizontal and vertical components of the absolute movement of a joint two separate applications of the method are generally required for each joint of a truss where the deflections of all joints are required to be known, whereas one graphical solution using the Williot-Mohr method gives the resultant deflections of all the joints of a truss with a considerable saving of time and labour. The last chapter is concerned with influence lines for simple frames. The book is illustrated with numerous clear diagrams, and students, particularly those working alone, will appreciate the large number of worked examples and problems with answers.

Books Received

"The Fundamental Principles of Reinforced Concrete Design." By W. T. Marshall. (London: Blackie & Son Ltd. 1951. Price 20s.)

"The Predetermination of Water Requirement and Optimum Grading of Concrete under Various Conditions." By N. M. Plum. Building Research Study No. 3 of the Danish National Institute of Building Research. (Copenhagen: Teknisk Forlag. 1950. Price 15 kr.)

"Architecture as a Career." By Maurice E. Taylor (London: Iliffe & Sons, Ltd. 1951. Price 10s. 6d.)

"Steelwork in Building." By W. B. Scott. (London: E. & F. N. Spon, Ltd. 1952. Price 25s.)

"Civil Engineering Plant and Methods." By Rolt Hammond. (London: Ernest Benn, Ltd. 1952. Price 25s.)

Shell Roof at a Store Building.

THE two main buildings of the new Post Office Stores at Kidbrooke, London, S.E., shown in *Fig.* 1, have a total area of 59,000 sq. ft. and are covered with reinforced concrete barrel-vault shell roofs. The standard dimensions of a bay are 50 ft. by 24 ft., and at each end of the buildings, adjacent to the expansion joints, the shell projects as a cantilever of nearly 17 ft. This column spacing was used mainly to avoid existing footings and to avoid underpinning existing walls. It also reduces and equalises the longitudinal bending moments on the barrels.

The barrel-vault shells have a radius of 20 ft. and a least thickness of 2 in. The concrete in the shell is a 1 : 1½ : 3

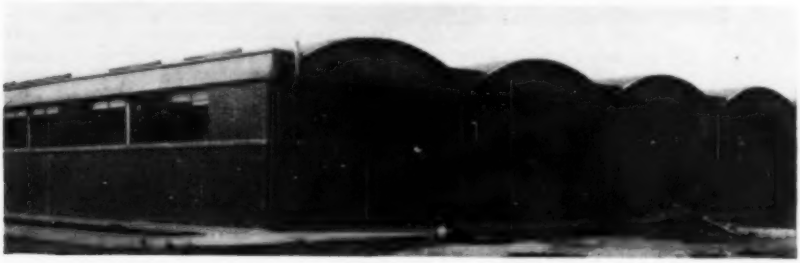


Fig. 1.

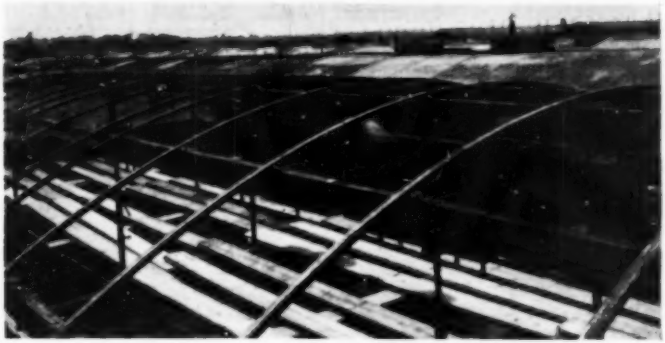


Fig. 2.—Centering for the Vaults.

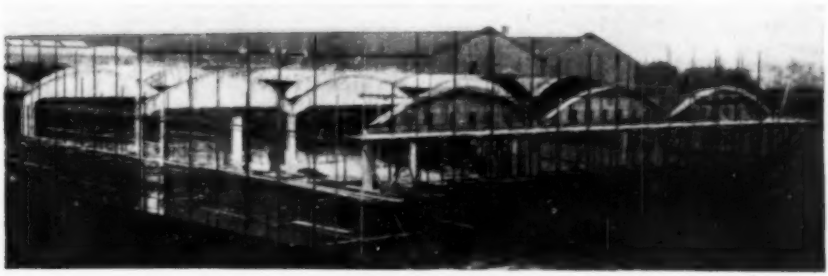


Fig. 3.—Diaphragms Cast in Advance of the Shells.

Fig. 4.

mixture with ¾ in. aggregate, and was cast on curved steel-sheet centering (*Fig.* 2). The diaphragms, cast in advance of the shell construction, are shown in *Fig.* 3. The soffits of the valleys between the intermediate barrels were formed with a bull-nose of precast lightweight concrete. The main reinforcement of the edge and stiffening beams is of square twisted high-tensile steel, and the reinforcement of the shells is high-tensile welded mesh with mild steel bars diagonally. The total weight of steel used was about 3 lb. per square foot of floor area.

The top lighting consists of 3 ft. 6 in. square lights, giving a glazed area of 7 per cent. of the floor area; the intensity of light is even and amply sufficient for most operations (*Fig.* 4). Insulation of the top surface of the roof was with a screed of vermiculite concrete 1½ in. thick, waterproofed with two layers of bituminous felt; this has been successful in preventing condensation on the soffit of the shells.

The architects were the Ministry of Works Maintenance Division and the consulting structural engineer was Mr. C. V. Blumfield. The contractors were Messrs. James Carmichael (Contractors), Ltd.

Conditions of Service in the County of Durham.

THE dispute between the Durham County Council and its professional employees, referred to in our June number, has been settled. The dispute arose over a requirement of the Council that all its employees must make any claim for extended sickness benefit through a trade union or professional society. Teachers, doctors, engineers, and other professional employees of the Council objected to this requirement, and after protracted negotiations both sides to the dispute agreed to abide by the decision of a board of arbitration appointed by the Minister of Labour. The arbitrators decided that this requirement should be withdrawn by the Council, and the Council has agreed to abide by this decision.

Copper Strips for Expansion Joints.

A BRITISH STANDARD (No. 1878) for Corrugated Copper Jointing Strip for Expansion Joints for use in General Building Construction (excepting large water containing structures and slabs carrying wheeled traffic) has been issued by the British Standards Institution. The quality of material and the dimensions are described. Widths of strips are 8 in. and 10 in.; the depth of the corrugation is 1¼ in. in the case of 8-in. strips and 1½ in., 1¾ in., and 2 in. in strips 10 in. wide. The radii of the bend at the root of the corrugation and from the corrugation to the flange must be ¼ in. Thicknesses are 0·022 in., 0·028 in., and 0·032 in., and the material is to be supplied in lengths of 8 ft. and 12 ft. unless otherwise ordered. The price of the document is 2*s*.

Design Loads for Buildings.

REVISION OF B.S. CODE NO. 3, CHAPTER V, "LOADING".

A REVISED edition of British Standard Code No. 3, Chapter V, "Loading," was published (price 4s.) last month by the British Standards Institution. The more important revisions are summarized in the following.

DEAD LOADS.—To provide for partitions where their position is not shown on the plans, beams (and the floor slabs where these are capable of effective lateral distribution of the load) must be designed to carry, in addition to the other loads, a uniformly-distributed load per square foot of not less than 10 per cent. of the weight per linear foot of the finished partitions, and in no case less than 20 lb. per square foot if the floor is to be used for offices.

SUPERIMPOSED LOADS ON FLOORS.—*Table* 1 gives the revised superimposed design loads for floors and *Table* 2 the reduction in superimposed loads for columns, walls and foundations of multiple-story buildings. No reduction may be made for factories and workshops designed for less than 150 lb. per square foot, or for warehouses, garages, or buildings used as stores. (These tables have been prepared by Mr. C. E. Reynolds, B.Sc., A.M.Inst.C.E.)

GARAGES.—The floors of garages for vehicles not exceeding 4 tons gross weight must be designed for (i) a uniformly-distributed load of 150 lb. per square foot or (ii) the most adverse combination

TABLE 1.

CLASS NO. AND SUPERIMPOSED LOAD (LB. PER SQUARE FOOT)		30	40	50	60	80	100	150	200
MINIMUM TOTAL LOAD (LB.)	SLABS (PER FOOT WIDTH)	240	320	400	480	640	800	–	–
	BEAMS	1920	2560	3200	3840	5120	6400	–	–
RESIDENTIAL BUILDINGS	Houses: Not more than two stories, one occupant					Class No. 30			
	Others (including flats)					. . 40			
	Hospital wards: dormitories					. . 40			
	Hotels: Bedrooms and private sitting-rooms					. . 40			
	Public rooms					. . 100			
COMMERCIAL BUILDINGS	Offices: Entrance floor and below					. . 60			
	Above entrance floor					. . 50			
	Storage and filing					. . 100			
	Banking halls					. . 60			
	Retail shops					. . 80			
	Book and stationery stores					. . 200			
	Warehouses (dependent on loads)					Class Nos. 100, 150, or 200			
INDUSTRIAL BUILDINGS	Workshops and factories (dependent on loads)					. . 100, 150 or 200			
	Workrooms (general)					Class No. 80			
	Light workrooms without storage					. . 50			
	Power stations and machinery halls; circulation space					. . 80			
PLACES OF ASSEMBLY	School classrooms					. . 60			
	Without fixed seating: dance halls					. . 100			
	With fixed seating: churches, restaurants					. . 80			

TABLE 2.

NUMBER OF FLOORS SUPPORTED		1	2	3	4	5 OR MORE
PROPORTION OF SUPERIMPOSED LOAD ON ALL FLOORS SUPPORTED	General	1·0	0·9	0·8	0·7	0·6
	Factories and workshops designed for 150 lb. per square foot or more	1·0	0·95	0·9	0·85	0·8

of actual wheel loads, whichever gives the greater stresses.

SUPERIMPOSED LOADS ON ROOFS.—For roofs sloping at less than 10 deg. and to which there is access the design load is to be 30 lb. per square foot with a minimum total load of 240 lb. per foot of width on slabs and 1920 lb. on beams. If there is no access to the roof the design load may be reduced to 15 lb. per square foot. On roofs sloping at more than 10 deg. but not more than 30 deg. the superimposed design load is 15 lb. per square foot, and on roofs sloping at 75 deg. or more no superimposed load need be considered. For slopes between 30 deg. and 75 deg. the load may be obtained by interpolation.

WIND PRESSURE ON BUILDINGS.—The requirements for wind loading have been changed and reference should be made to the code for details.

[The design data and tables in "Examples of the Design of Reinforced Concrete Buildings," by C. E. Reynolds (see p. 276), are based on the loadings given in this code.]

Deformation of Floor Slabs.

IN Proceedings No. 15 of the Swedish Cement and Concrete Institute an account is given of an investigation into the effects of construction loads on suspended concrete slabs. Frequently the weight of a newly-cast slab is transmitted by the centering and props to recently-cast slabs below which have not reached their full strength and which consequently may have considerable permanent deformation. If the loads are great enough to crack the concrete the decrease in flexural rigidity of the slab results in greater deflections under later loading. A method of calculating the loading and bending moments is given and is illustrated by a typical example.

Tests were made to determine the loads occurring during the construction of a seven-story block of flats and to assess the effects on these loads of variations in the moisture content of timber centering. The following results were obtained from the tests.

(1) The plastic deflections produced during the period of construction are three to six times as large as the elastic deflections (calculated as the deflections caused by the dead weight alone at the time of removal of the centering from the floor above). (2) The deflections of simply-supported floor slabs are about twice as large as those of slabs fixed at the sides at approximately equal loads during the period of construction. (3) The plastic deformations become greater as the loads acting on slabs increase and as these loads occur earlier, that is as the rate of construction increases.

These results are in close agreement with the theoretical treatment of the loads and bending moments, but the deformations were several times as large as those computed. The following inferences are drawn from the theoretical treatment. (a) The deformations increase in approximately inverse proportion to the square of the thickness of the slab. (b) The deformations increase approximately as the fourth power of the span. (c) The risk of cracking and the resultant decrease in flexural rigidity are greater in simply-supported floor slabs than in slabs fixed at the sides. This is important in a comparison between these two methods of support, since the loads causing cracks may be approached or exceeded during construction. (d) The deformations decrease as the elastic rigidity and the plastic rigidity of the centering become greater. (e) The deformations decrease as the rigidity—particularly the plastic rigidity—of the base carrying the centering for the upper floor slab becomes greater. If the moisture content of timber centering immediately before placing the concrete is low, and drying of the timber after casting the slabs is limited, the deformations are reduced.

The report recommends that slabs should be cast with adjacent walls where these are of concrete and that the centering, which should be as rigid as possible, should be kept in position as long as practicable.

Bridge with Adjustable Prestressing Cables.

THE bridge at Villeneuve-Saint-Georges, near Paris, shown in *Fig.* 1, has three spans of 134 ft., 256 ft., and 134 ft., the middle span consisting of two cantilevers with a freely-supported central section 129 ft. long. The bridge is of hollow construction with three longitudinal compartments. The roadway is 27 ft. wide with cantilevered footpaths 9 ft. wide on each side (*Fig.* 2). The deck is supported on concrete hinges at the intermediate piers and on rockers at the abutments. The middle span is supported on a concrete hinge at one end and a concrete rocker at the other (*Fig.* 1).

The deck is prestressed in a longitudinal direction, and the amount of the prestress can be adjusted from the interior of the bridge by means of an arrangement of wire cables and short struts hinged at their feet and over which the bent-up cables are placed. Man-holes are left in the deck to provide access to the interior, where electric lighting is installed.

The profiles of and the forces in the cables have been selected to achieve the following two conditions for each section of the bridge : (1) Under the action of the dead load the resultant of all the forces and reactions at the supports pass near the centre of gravity of the cross section, that is, the precompression should be equally distributed over the whole cross section. (2) Under the action of the dead load and the worst combination of live loads the longitudinal precompression should never be less than 70 lb. per square inch in order to avoid cracking of the concrete. To achieve these conditions eight straight and sixteen bent-up cables were required near the bottom of the simply-supported middle span and thirty cables were placed near the top over the two support piers. Some of the cables in the cantilevers were bent down to reduce the shear stresses.

The arrangement for the adjustment of the forces in the cable has been made for

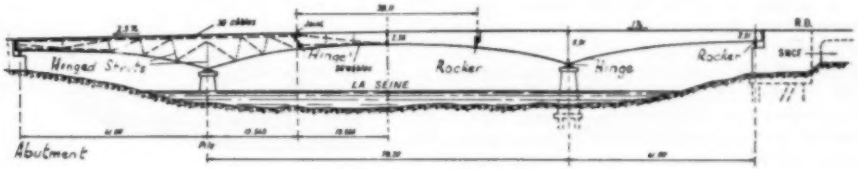


Fig. 1.—Elevation.

two main reasons : (*a*) The tensile forces can be increased as required to allow for shrinkage and creep of both concrete and steel. This renders the behaviour of the structure practically independent of these factors, which at best can only be calculated approximately in advance. (*b*) Any future increase in the allowable superimposed load can be carried by increasing the tension in the cables.

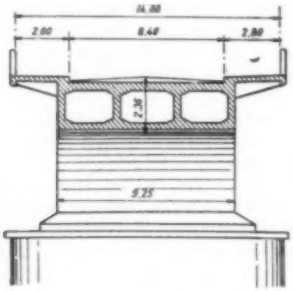


Fig. 2.—Cross Section.

Composition of Cables and Anchorages.

Each cable consists of 193 wires of 4·1 mm. diameter with an ultimate tensile strength of 100 to 115 tons per square inch and a total tensile force of 160 metric tons, producing a stress of 40 tons per square inch which, with the addition of the superimposed load, is nearly 41 tons per square inch. Each cable is provided with a steel anchoring device which consists principally of four bars of 2½ in. diameter provided with nuts bearing against a fixed thrust-block and a movable anchor plate.

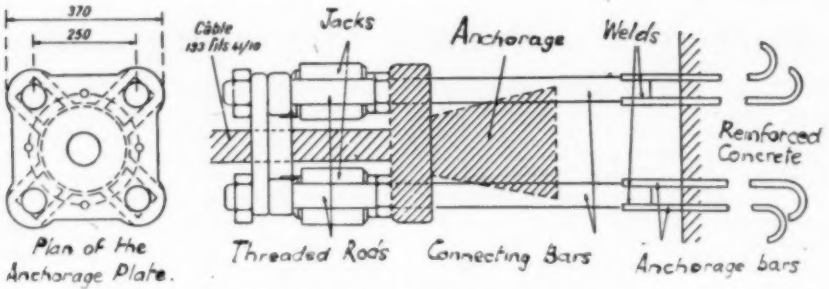


Fig. 3.—Method of Anchoring Cables.

Welded on to each bar are three 1¼-in. diameter bars cast in the concrete so as to transfer their load uniformly (*Fig.* 3). The cables were tensioned by four hydraulic jacks per cable acting between the movable anchor plate and the fixed thrust-block.

Where cables are bent up or down they are taken over or under short struts hinged at their bases. In the simply-supported central span, where the overall depth of the bridge is small, these struts are of steel (*Fig.* 4). In the other portions they are of concrete, strongly reinforced, with a hinge at the bottom and a grooved steel plate at the top to avoid crushing the concrete

Fig. 4.—Hinged Steel Strut.

Construction.

The requirements for navigation were that a free passage had to be left and for this reason the outer spans were constructed first. The middle portion was then built from suspended metal centering supported on the two piers and on a trestle at the centre. The pathways were constructed last. *Fig.* 5 shows in diagrammatic form the main phases of the operation.

As the weight of the independent span increased during construction, high stresses in the cantilevers were avoided by adjustment of the forces in the cables. Two graphs were drawn giving values between which the tension had to be maintained at all times. The tensioning was carried out in steps of 15 tons per cable to reduce the number of stressing operations. The movements of the hinged struts were closely investigated and their final positions agreed very well with those calculated when the maximum force of 160 tons was reached. Four hydraulic jacks were sufficient for the whole prestressing operations.

Materials.

Cements of high early strength were not used on account of their shrinkage and the mixture had a cement content of 675 lb. per cubic yard of finished concrete. Test specimens were taken for tests at 7, 28, and 90 days, twenty-one specimens being taken from each 10 cubic metres of concrete. At 90 days the average compressive strength was 5000 lb. per square inch on prisms and 6330 lb. per square inch on cubes.

Six tensile tests were made on short lengths of cable attached to their anchors. The results obtained varied between 394 and 420 tons per cable, giving a factor of safety of 2·5. All cables were subjected on a trial bench to a tension of 200 tons, which is 25 per cent. more than the maximum normal force. From these tests the elastic elongation after final tensioning was decided.

In order to check the stresses due to the

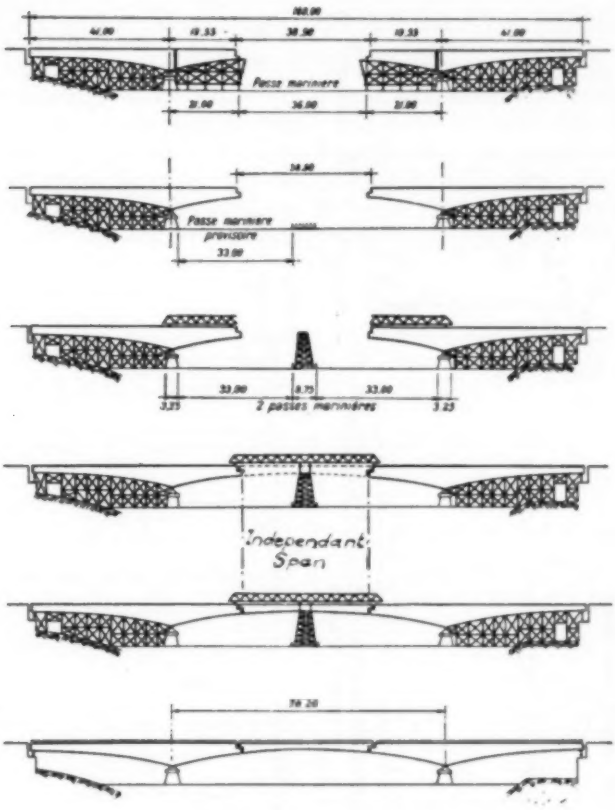


Fig. 5.—Main Phases of Construction.

weight of the beams and of the tensioned cables, 100 acoustic extensometers were built in the longitudinal and transverse beams. Extensometers were also fixed to the cables to check the gauge readings of the hydraulic jacks.

The finished bridge was test loaded and showed a deflection of 13 mm. for the central span, or 1/6000 of its span. This was less than that calculated for a value of E of 5,000,000 lb. per square inch.

The design was by M. Henry Lossier in collaboration with the engineers of the Service des Ponts et Chaussées.

A Symposium on the Chemistry of Cement.

AUTHORITIES from many countries will present papers at a symposium on the chemistry of cement to be held in London from September 15 to 19. The symposium has been arranged by the Cement and Concrete Association (52 Grosvenor Gardens, London, S.W.1), from whom full details can be obtained.

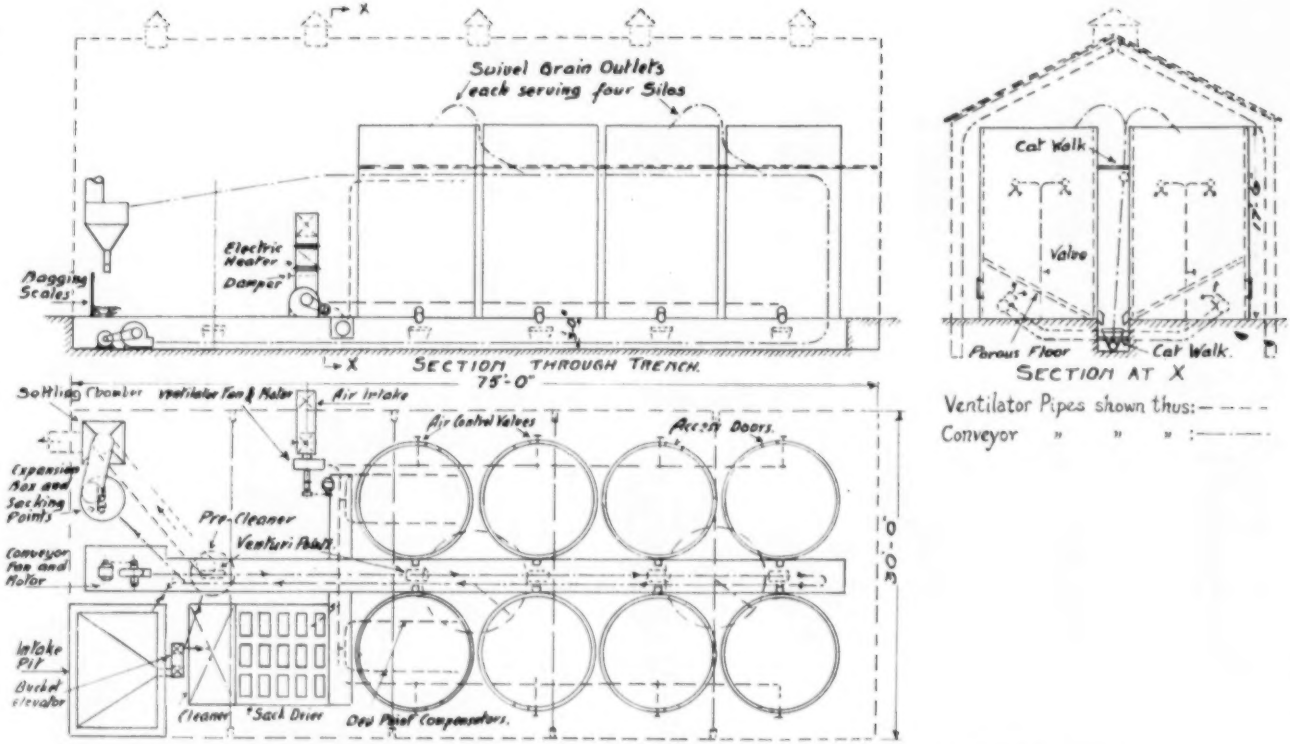


Fig. 2.—Diagrammatic Arrangement of Electric and Pneumatic Grain-conditioning Plant.

(*See facing page.*)

Silos for Storing Grain on the Farm.

By A. M. PENNINGTON.

As about 18,000 combine harvesters will be at work in Great Britain for this year's harvest, and as much grain was spoilt last year due to lack of storage, the question of drying and storing grain is of urgent importance. The concrete-stave silo as used for making silage is the cheapest and most adaptable means of storing grain if independent drying plants are used. In the warm-air process each silo must have a porous false bottom which permits the passage of air but retains the grain. The silos are generally from 15 ft. to 18 ft. high and 10 ft. to 14 ft. diameter, which allows for two rows of silos with a space between them in the barn

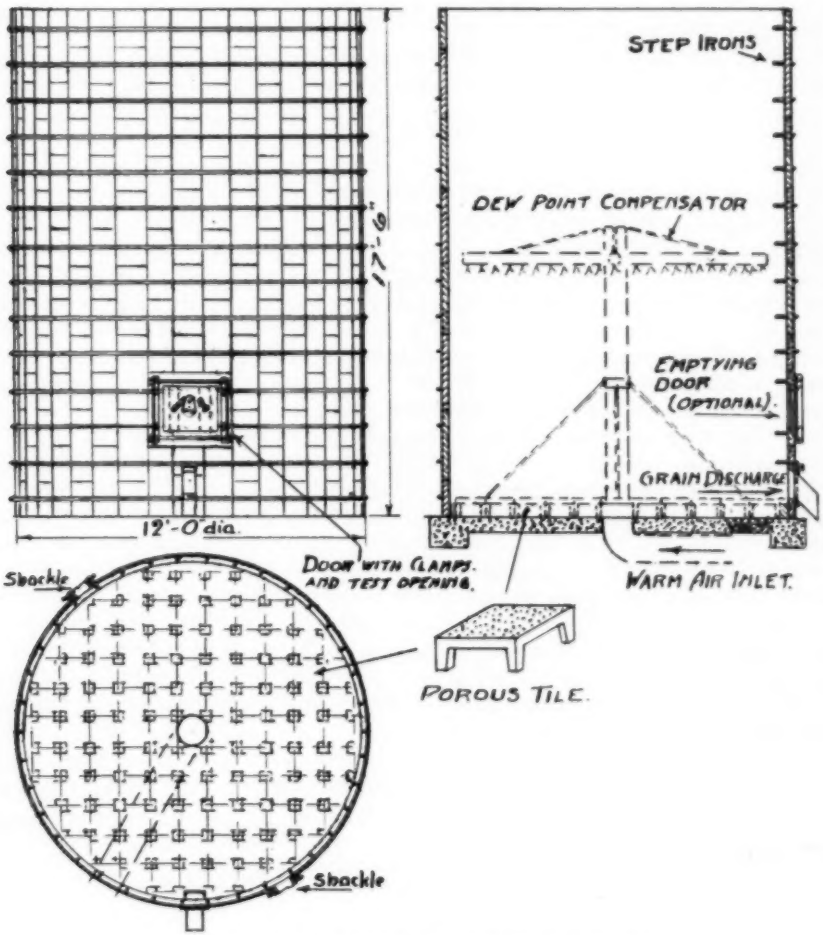


Fig. 1.—Typical Stave Silo for Drying Grain.

(generally about 30 ft. wide by 15 ft. to 18 ft. high) in which the drying plant and silos are installed. The silos must be protected from the weather, and the joints between the staves airtight ; it is best to bed or point the staves in 1 : 1 : 6 cement-lime-sand mortar. Large ventilators or extractor fans are required in the roof of the barn to remove the moist air rising from the tops of the silos during the drying process. Access to the tops of the silos and the filling-pipes is by a cat-walk between the silos at high level, from which the silos can be entered by means of rungs formed of mild steel bars cast into a vertical line of staves. When the silos have a flat false bottom, as in *Fig.* 1, the last of the grain has to be shovelled towards the discharge pipe by hand. Sometimes the action is assisted by inclining the bottom towards the pipe (*Fig.* 2—see p. 284), but the extra expense and loss of capacity are rarely justified. Where an independent dryer is installed the silo is a simple bin with an outlet but without the false bottom and the warm-air inlet and dew-point compensator.

A typical arrangement of a farm grain-drying plant is seen in *Fig.* 2, where the grain from the harvester is emptied into a pit from whence it is elevated to a cleaner which removes chaff, leaves, and insects. As the grain may arrive faster than it can be properly cleaned, it may be diverted to a pre-cleaner which quickly removes most of the debris. From the cleaner the grain enters (by way of a venturi point which works on the ejector principle) the conveyor-pipe through which a fan is continuously blowing air. Through this pipe the grain is lifted and may be blown into any of the silos ; from the upper part of the conveyor-pipe two branches are each capable, by means of a swivel outlet, of supplying any one of four silos, the grain being diverted where required by valves in the pipe. The part of the conveyor-pipe in the trench is provided with venturis and receiving hoppers opposite each silo discharge-pipe so that, by means of valves, the grain may be returned to the intake hopper to be passed through the cleaner, and transferred to any silo or to the sacking point.

Filtered air is drawn into the ventilator fan after passing through a chamber containing an electrical resistance heater. The warmed air is blown along the pipes, through the porous floors, and through the grain, carrying the moisture with it. A supplementary supply passes through the dew-point compensators and is discharged at a suitable height in the silos. This is necessary if the depth of grain is more than 7 ft. because the warm air becomes laden with moisture and cools as it rises, and when the dew-point is reached (at a height of about 7 ft.) the moisture would be deposited in the cooler layers. A further supply of warm air is directed to the sack-dryer, which is a low hollow structure with openings on the top upon which sacks of grain are placed so that the warm air admitted to the interior of the structure under slight pressure escapes by passing through the sacks, carrying any moisture with it.

Design of Silo.

The lateral pressure p in lb. per square foot on the wall of a circular storage bin is given by Janssen's formula :

$$\frac{wR}{\mu_1}\left(1 - e^{\frac{-k\mu_1 H}{R}}\right) \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where R is the hydraulic radius of the bin in feet, w the weight per cubic foot of

TABLE I.

Material	Weight (lb. per cu. ft.)	Rough boards		Smooth boards		Steel		Concrete	
		μ_1	k	μ_1	k	μ_1	k	μ_1	k
Wheat	55	0·450	0·450	0·400	0·38	0·468		0·444	0·6
Barley	40	0·424	0·450	0·325	0·38	0·376		0·452	
Oats	35	0·450	0·450	0·369	0·38	0·412		0·466	
Shelled corn (maize)	44	0·344	0·450	0·308	0·38	0·374		0·423	
Soya beans	46	0·435	0·450	0·322	0·38	0·366		0·442	
Flax seed	43	0·407	0·555	0·308	0·50	0·399		0·414	
Rye	46	0·540	0·300	0·460	0·30			0·850	0·3

the material stored, μ_1 the coefficient of friction of material on the wall of the bin, k the ratio of lateral to vertical pressure, H the depth of material in feet, D the diameter of the bin in feet, and e the base of Naperian logarithms = 2·718.

Since $R = \frac{D}{4}$ the formula may be written

$$p = \frac{wD}{4\mu_1}\left(1 - e^{\frac{-4k\mu_1 H}{D}}\right) \quad . \quad . \quad . \quad . \quad (2)$$

Table 1 gives values of the constants for various materials. This table is not complete, but as a silo may be used to store various grains it will suffice if it be designed to store the heaviest grain, namely, wheat. The curves in *Fig.* 3 have been plotted from Janssen's formula with the constants for wheat given in *Table* 1

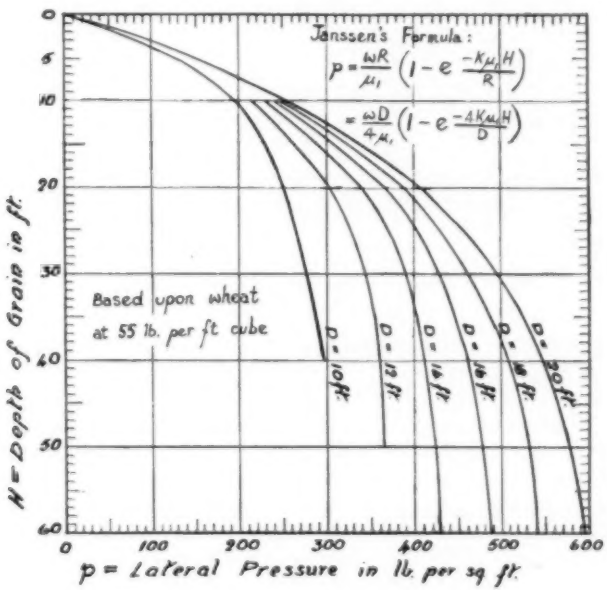


Fig. 3.—Pressure on Wall.

under "Concrete"; the weight of wheat is assumed to be 55 lb. per cubic foot, which represents exceptionally heavy wheat. Inserting the values for wheat, the formula becomes

$$p = 31D\left(1 - 2{\cdot}718^{\frac{-1{\cdot}066H}{D}}\right).$$

The curves given in *Fig.* 4 are derived from those in *Fig.* 3 by applying the formula $P = pD = 2Af$ (see *Fig.* 5).

$$\therefore A = \frac{pD}{2f} \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where p and D have the meanings already defined, A is the area of the steel in square inches required in the bands per foot height of silo at various depths from

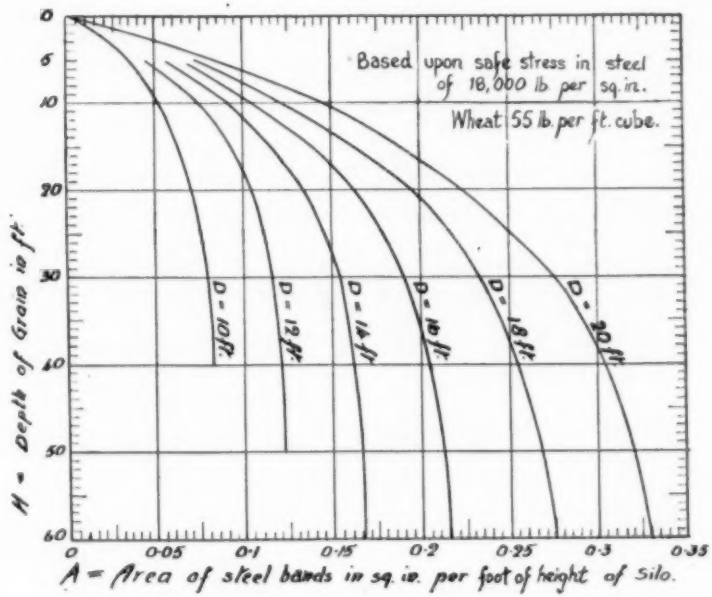


Fig. 4.—Bands Required at Various Depths.

the top, and f is the safe stress in the steel which is assumed to be 18,000 lb. per square inch at the weakest point, namely the bottom of the thread.

Table 2 is derived from *Fig.* 4 and gives the best spacing at various depths from the top (of silos of various diameters) of steel bands $\frac{1}{2}$ in. diameter and having a B.S.F. thread, or alternatively $\frac{7}{16}$-in diameter bands having a $\frac{1}{2}$-in. diameter B.S.F. thread rolled on. *Table* 3 is similar to *Table* 2, but the spacing of the bands at various depths is given to the nearest suitable factor of 30 in., the height of the stave (*Fig.* 8). The figures in brackets indicate that at the top of the smaller silos only one band per stave is necessary; at the bottom of the larger silos there are six bands to each stave. It is a simple matter to find from *Table* 2 the band-spacing for any other height of stave. In *Table* 4 the spacing is shown for bands

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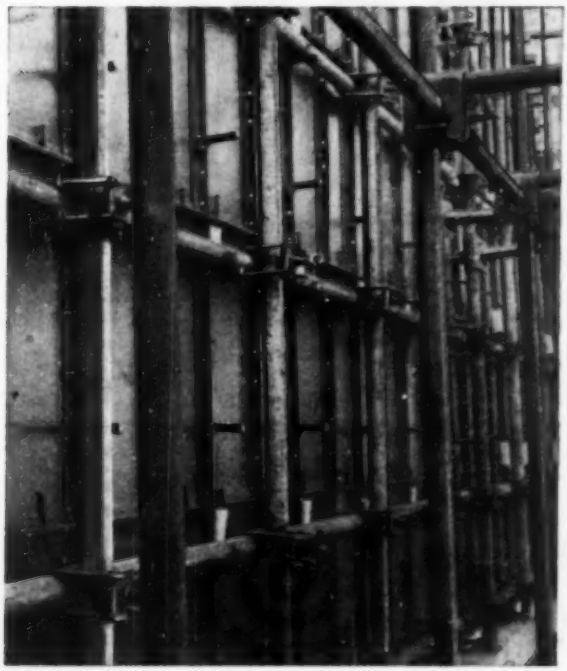

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TABLE 2.

Distance of bands below top of silo	Diameter of silo in feet					
	10	12	14	16	18	20
Threads $\frac{1}{2}$ in. B.S.F.	Band spacing in inches					
5	48·4	37·7	30·0	25·5	23·0	20·5
10	31·5	22·4	18·5	15·8	13·8	11·6
20	23·6	15·8	12·5	10·0	8·5	7·5
30	21·2	14·4	10·7	8·7	7·1	6·0
40	20·2	13·7	10·2	8·1	6·5	5·43
50		13·5	10·0	7·8	6·2	5·18
60			9·9	7·65	6·0	5·0

TABLE 3.

Threads $\frac{1}{2}$ in. B.S.F.						
5	15(30)	15(30)	15(30)	15·0	15·0	15·0
10	15(30)	15	15·0	15·0	10·0	10·0
20	15	15	10·0	10·0	7·5	7·5
30	15	10	10·0	7·5	6·0	6·0
40	15	10	10·0	7·5	6·0	5·0
50	15	10	10·0	7·5	6·0	5·0
60	15	10	7·5	7·5	6·0	5·0

TABLE 4.

Threads $\frac{9}{16}$ in. B.S.F.						
5	15(30)	15(30)	15(30)	15	15·0	15·0
10	15(30)	15	15	15	15·0	15·0
20	15	15	15	15	10·0	10·0
30	15	15	15	10	7·5	7·5
40	15	15	15	10	7·5	6·0
50	15	15	10	10	7·5	6·0
60	15	15	10	10	7·5	6·0

having $\frac{9}{16}$-in. cut or rolled threads. In the latter case $\frac{1}{2}$-in. diameter bars are used. Above the dotted lines $\frac{1}{2}$-in. B.S.F. threads may be used as in *Table* 3.

Fig. 6 shows a malleable iron or steel shackle used for connecting the ends of the bands. To ensure equal distribution of stress throughout the length of the band, two bars and two shackles may be used for silos up to 10 ft. diameter, three

Fig. 5.

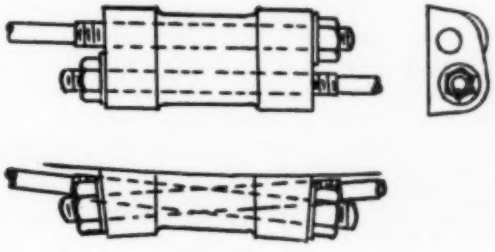

Fig. 6.

for silos from 10 ft. to 16 ft., and four from 16 ft. to 20 ft. Distribution of stress is also assisted if, after tightening, each band is struck with a mallet at the mid-point of each stave.

Although not essential, it is a convenience to have an emptying door in each silo so that in the event of a breakdown, or of mould forming and preventing the flow of grain, the silo may be emptied through the door. A refinement is to have a bagging-spout cast on the door so that small quantities of grain may be taken out of the silo without operating the whole plant (*Fig.* 7). The bands may be bolted to the door-frame as shown or trimmed around the door as in *Fig.* 1. The trimmer-bolts must be strong enough to transmit the full load on all the bands

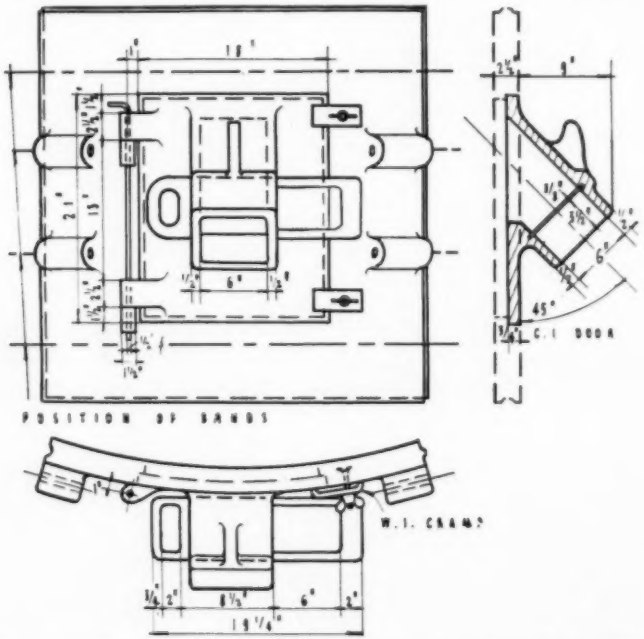

Fig. 7.—Cast-iron Door with Bagging-spout for Staves 3 ft. high. The height of the Door would be 2 ft. 6 in. with 2 ft. 6 in. Staves.

attached to the angle-irons. In the case of large silos there may be more than two bands as shown in *Fig.* 1. Owing to the exposed position of the bands, to the prestress when they are tightened, and to possible over-stress due to temperature changes, it would seem wise to have them on the large side. A sudden drop in air temperature would increase the stress in the bands since, owing to its greater thermal conductivity, the steel would contract before the staves. Even so, in the U.S.A. a working stress of 20,000 lb. per square inch is used for bands on grass silos in the open air.

Staves.

Fig. 8 shows the staves now favoured in this country. Whole staves are 30 in. by 12 in. by 2½ in. thick, and half-staves are 15 in. by 12 in. by 2½ in. thick.

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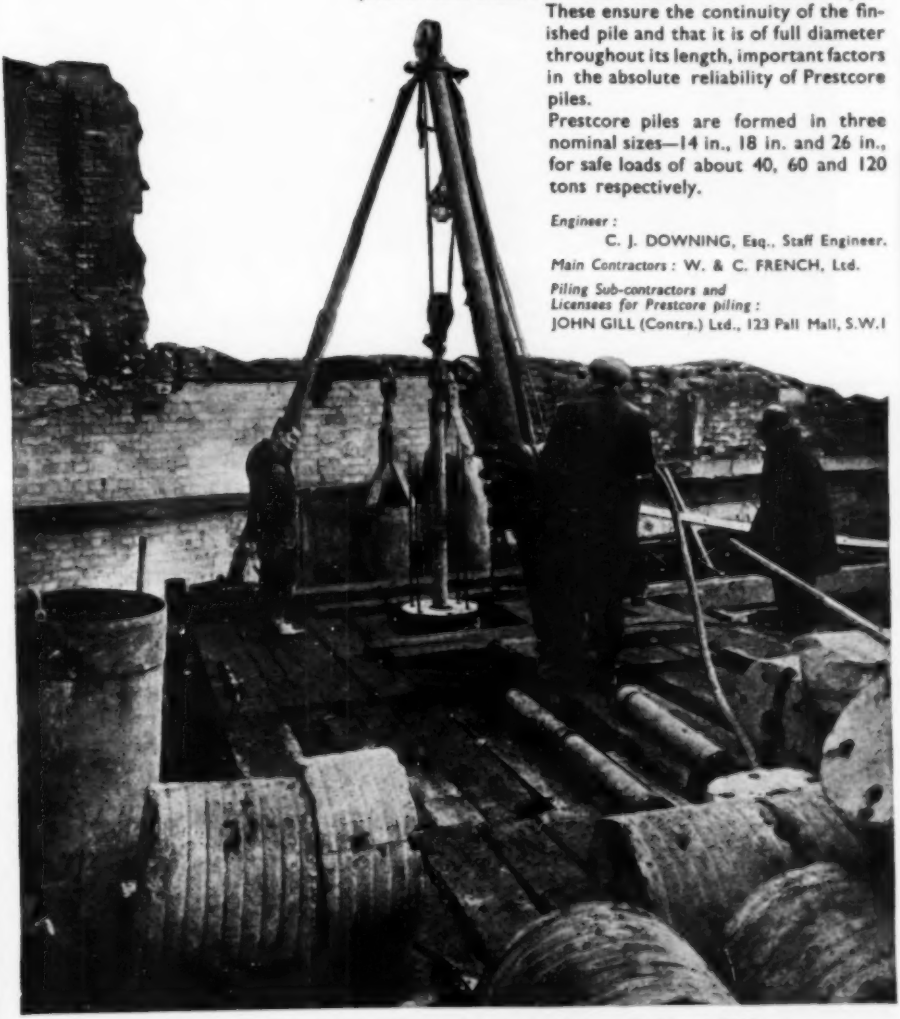

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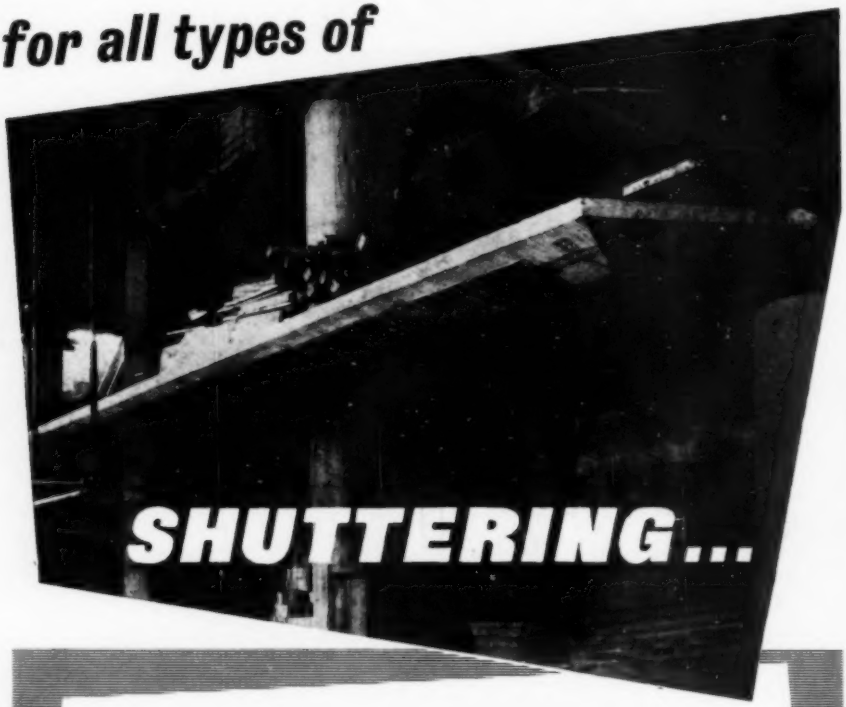

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The staves are generally hydraulically pressed or made of 1 : 4 plastic vibrated concrete in wooden or steel gang moulds, and are not reinforced. They are tongued on two adjacent sides and grooved on the other two sides. The vertical sides are tapered radially (see section) so that the staves may be used for a range of silo diameters without having gaping joints. It is advisable to make the staves with the narrow faces upwards so that they may be trowel-finished and thus have a smooth interior surface. When erecting a silo the bottom ring of staves is set up, consisting of alternate whole and half staves with the grooves at the bottom and the narrow faces inwards. There must be an even number of staves in each ring. The joints (*Fig.* 8) are designed to allow a range of diameters from 10 ft. to 20 ft. The lowest bands are fixed and temporarily tightened. Further whole staves and bands are added until the top row is reached, where a ring of alternate

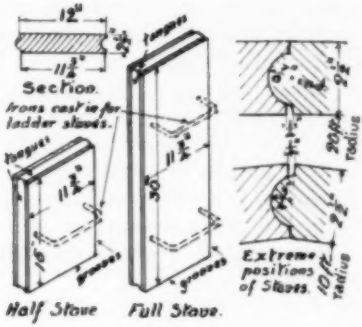

Fig. 8.

whole and half staves brings the top level. The bands may be finally tightened after the joints are pointed and the mortar has hardened.

In the absence of a British standard specification, the following requirements for the staves are suggested. (1) Absorption after submersion in water for 24 hours should not exceed 6 per cent by weight. (2) The staves should be tested for strength immediately after submersion for 24 hours, the surface-water being wiped off with a damp cloth. The stave should be supported on 2-in. diameter steel bearers on a 24-in. span, the supports being free to rotate parallel and perpendicular to the long axis. The load should be applied centrally at a uniform rate, starting from zero and not exceeding 150 lb. per minute, on a steel bearer 2-in. diameter midway between the supports. A suitable apparatus for the transverse test would be similar to that used for paving slabs (B.S. No. 368/36) or, better still, a more substantial form of that used for roofing tiles. For staves 2½-in. thick the average breaking load of five staves should not be less than 120 lb. per inch of width, and not less than 100 lb. for any stave.

With the minimum breaking load of 100 lb. per inch of width the bending moment on a stave 2½ in. thick would be $\frac{wl}{4} = \frac{100 \times 2}{4} = 50$ ft.-lb. per inch of width. Allowing a factor of safety of 3, the permissible safe bending moment on a stave 12-in. wide would be

$$\frac{50 \times 12}{3} = 200 \text{ ft.-lb.} \quad . \quad . \quad . \quad . \quad (4)$$

For proper support each stave must have at least two tension bands, so that the maximum spacing of the bands is 15 in. for a 30-in. stave. If such a stave be supported by bands l ft. apart and $\frac{l}{2}$ ft. from each end, then, neglecting any support from adjacent staves, the maximum bending moment is $\frac{wl^2}{8}$ and occurs at the supports (w is the distributed load per foot of length of the whole stave which, on staves 12 in. wide, is the same as the load per square foot). Substituting $M = 200$ ft.-lb. and $l = 1{\cdot}25$ ft., then $w = \frac{8M}{l^2} = \frac{8 \times 200}{1{\cdot}25^2} = 1025$ lb. per square foot for a 30-in. stave. *Fig.* 3 shows that the lateral pressure at the base of a silo 20 ft. diameter and 60 ft. high is nearly 600 lb. per square foot, so that safe stresses due to bending alone are not exceeded for such a silo even if the bands be 15 in. apart at the bottom. With such a spacing some of the bands would, of course, be larger in diameter than those given in *Tables* 2 to 4. Compressive stresses due to the weight of the wall and of the part of the contents transmitted to the wall by friction, and the horizontal compressive stress due to the tightening of the bands, vary with the height and diameter. In the worst case under consideration, a silo 60 ft. high and 20 ft. diameter (*Fig.* 3), the lateral pressure at the bottom when filled with wheat is about 600 lb. per square foot. The pressure exerted by the bands, if the joints are not to open, must be greater than the lateral pressure exerted by the grain, the excess band-stress being resisted by the staves in horizontal compression.

Considering the lateral compressive stress in the stave, if the safe compressive stress in the steel f_s is assumed to be 18,000 lb. per square inch and in the concrete f_c is 950 lb. per square inch, from *Fig.* 4 the area of band-steel required at the base of a silo 60 ft. high and 20 ft. diameter is 0·33 sq. in. per foot, then the actual compressive stress in the concrete is $f_c \times 12 \times 2\frac{1}{2} = 18{,}000 \times 0{\cdot}33$, from which the compressive stress in the concrete is 200 lb. per square inch, which is well below the safe stress. The staves are subjected to this lateral compressive stress only when the silo is empty. When there is a pressure due to grain, the bands extend and the compression of the staves is reduced to the residual amount due to the initial tightening of the staves; this is difficult to calculate but so small that it may be ignored.

The vertical compressive force on the staves is that due to the weight of the superimposed wall plus the friction of the grain. Assuming the weight of the concrete to be 150 lb. per cubic foot, the height of the silo H ft., the diameter D ft., and the thickness of the staves t in., the whole weight W of the silo is

$$\frac{150\pi DHt}{12 \times 2240} = \frac{DHt}{57} \text{ tons, or, if } t = 2\tfrac{1}{2} \text{ in., } W = \frac{DH}{22{\cdot}7} \text{ tons.}$$

The lateral pressure on the wall due to the weight of grain is given in *Fig.* 3 for silos of various heights and diameters. This pressure multiplied by the coefficient of friction between the grain and the wall gives the downward pressure on the wall due to the grain. Since this pressure varies with the depth of grain, the sum of the pressures at various depths is the pressure on the bottom staves. This can be obtained from Airy's formula or from a formula derived from Janssen's

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word : minimum 7s. 6d. Situations Vacant, 4d. a word : minimum 10s. Other miscellaneous advertisements, 4d. a word : 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

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SITUATIONS VACANT. Senior and junior civil engineering draughtsmen-designers required for steelwork, reinforced concrete, and building work for electricity generating stations and general industrial work. The salary will be not less than £700 per annum for senior appointments, and will be under constant review. Full superannuation scheme in operation. Applications, giving full details of experience, etc., to C. S. ALLOTT & SON, CONSULTING CIVIL ENGINEERS, 1 North Parade, Manchester, 3.

SITUATION VACANT. Civil engineering assistant capable of designing and detailing reinforced concrete structures required by Edinburgh firm of consulting engineers. Details of qualifications, experience, and salary required to BOX 2589, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Chief designer required by consulting structural engineers, Westminster. Considerable experience of reinforced concrete buildings essential, and experience of structural steelwork desirable. Applications in confidence, stating age and qualifications, with full details of experience, to BOX 2590, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete designers required for consulting engineer's office in Westminster. One vacancy is for a senior position. Apply in writing, giving full details of age, training, and experience. BOX 2587, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting structural engineer, Westminster, requires senior designer-draughtsman, with first-class experience in reinforced concrete, for responsible position. Experience in structural steelwork an advantage. High salary and good prospects for suitable applicant. Write in confidence, stating age, qualifications, and full details of experience. BOX 2591, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Experienced engineer required for London office of reinforced concrete specialists. Salary £1,000 p.a. or according to experience. Write, with particulars, to BOX L.950, c/o Streets, 110 Old Broad Street, London, E.C.2.

SITUATION VACANT. Measurer required in estimating department in London of a company of reinforced concrete engineers and contractors. Age 25–30. Five-day week and pension scheme. Write, giving full particulars of age, education, and previous experience and salary required, to Box C.E. 481, at 191 Gresham House, London, E.C.2.

SITUATIONS VACANT. Ove Arup & Partners require several reinforced concrete designer-detailers. Apply in writing, giving full particulars of education, training and experience, and stating salary required, to above at 8 Fitzroy Street, London, W.1.

SITUATIONS VACANT. The Trussed Concrete Steel Co. Ltd., Truscon House, 35–41 Lower Marsh, London, S.E.1, require additional staff in their London and Manchester design offices : (*a*) Designer-Detailers (age 25 or over), (*b*) Detailer-Draughtsmen (age 20 or over). Previous good experience in reinforced concrete drawing office is important. Five-day week and pension scheme. Apply in writing to the above address giving full particulars of age, education, and previous employment.

SITUATIONS VACANT. Reinforcement manufacturers have vacancies in their Bristol office for reinforced concrete designers. Posts are available for applicants with good experience, and also for suitably qualified persons for training. Work involves a variety of structures. Please give fullest details when replying. BOX 2594, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Precast concrete factory, South Midlands, has vacancy for a production engineer, age 25–30, preferably married. Successful applicant will be required to supervise production of high quality precast concrete structural components. Considerable practical experience in precast concrete industry or high quality in-situ work necessary. Good educational and technical background essential. This appointment offers excellent opportunities for keen young engineer who wishes to specialise. Modern accommodation available after probationary period. Box 2593, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. A company specialising in the design of reinforced concrete buildings requires an engineer for site inspection of work carried out by contractors to the company's designs. Applicants should have held responsible positions as resident engineers or agents for reinforced concrete constructional work. Visits to sites in all parts of the country will be entailed. Some design experience is desirable. Write, giving details of age, experience, and salary required, to BOX C.E. 475, at 191, Gresham House, London, E.C.2.

NORTH THAMES GAS BOARD.—Senior draughtsmen are required in the Chief Engineer's department at Westminster : (*a*) Experienced in the design of steel structures ; (*b*) Experienced in the design of reinforced concrete structures and general civil engineering work. Starting salary range £700 to £825 per annum, according to age, qualifications, and experience. The appointments are of a permanent nature after a probationary period, and pension arrangements will be discussed with short-list candidates. Applications, giving age, qualifications, and experience, should be sent to the STAFF CONTROLLER, NORTH THAMES GAS BOARD, 30 Kensington Church Street, London, W.8, quoting reference 666/39

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers with specialist experience for their Stafford, London, and Newcastle-upon-Tyne offices. Five-day week and staff pension scheme. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

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SITUATION VACANT. Designer-draughtsman with several years' experience of reinforced concrete design and detailing required in London area. Must be keen, and require minimum of supervision. Interesting work. Permanent position. Write, giving details of age, experience, and salary required. BOX 2597, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT.—Reinforced concrete designer. Simon-Carves Ltd. have a vacancy in their London drawing office for a reinforced concrete designer with sound experience of industrial work. The firm specialises in the design and erection of power stations, coal preparation and chemical plant, and offers a wide range of interesting work. Salaries are above average and a pension scheme is in operation. Working conditions and scope are good. Apply to PERSONNEL DEPARTMENT (YM27), SIMON-CARVES LTD., Cheadle Heath, Stockport.

(*Continued on page* lxvi).

SITUATIONS VACANT. Imperial Chemical Industries Limited, Wilton Works, near Redcar, requires to augment its present drawing office staff for the development of its large new chemical factory. Permanent employment will be offered to selected draughtsmen in the following categories: Civil, Instrument installation, Mechanical, Structural. Congenial working conditions include membershlp of staff pension fund, recreation club facilities, etc. Further details of conditions of employment, pension fund, and housing will be forwarded on request, together with official application for employment form. Apply to the nearest Employment Exchange or Staff Officer, Imperial Chemical Industries Ltd., Wilton Works, P.O. Box 54, Middlesbrough, quoting advertisement reference ICI/X/114/f.

SITUATION VACANT. The Stanton Ironworks Company Limited, near Nottingham, proposes to appoint a representative to cover the British Isles in connection with the sale of prestressed concrete pipes. Applicants should have a technical background of training and experience, but the successful applicant will have full opportunity of studying the process of manufacture and familiarizing himself with the products. The appointment will involve interviewing at high levels. The post will be superannuated. Applications, in confidence, should be made, in writing, to the ASSISTANT MANAGING DIRECTOR, giving age and full details of qualifications and experience.

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Snowcrete, £12 14s. 6d., inc. paper bags.
"Super-Cement," 32s. 6d. per ton above ordinary Portland cement; paper bags included.
High-alumina cement, 1 ton and upwards, 292s. 6d. per ton.
Snowcem paint, 71s. per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1s. 1d.) and No. 94 (price 5d.) issued by H.M. Stationery Office.

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Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3s. 6d. per hour (carpenters 2d. a day tool money); Labourers, 3s. 0½d.; Men on mixers and hoists, 3s. 2½d.; Bar-benders, 3s. 3d.

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formula, but it is simpler and sufficiently accurate to calculate from *Fig.* 3 the result for any particular case.

Example.

Find the maximum vertical pressure on the wall of a silo 20 ft. diameter, 2½ in. thick, and 60 ft. high, filled with wheat. By interpolation in *Fig.* 3 the lateral pressures at depths D of 5, 15, 25, 35, and 45 ft. may be read from curve $D = 20$ ft. The sum of these multiplied by 10 and by the coefficient of friction from *Table* 1 gives the answer required.

Therefore the vertical pressure due to friction of the grain at the base of the silo per foot of wall is

$$\frac{10(145 + 340 + 460 + 525 + 565 + 590) \times 0{\cdot}444}{2240} = 5{\cdot}2 \text{ tons.}$$

To this must be added the weight per foot of circumference of the wall, namely,

$$\frac{150Ht}{12 \times 2240} = \frac{150 \times 60 \times 2{\cdot}5}{12 \times 2240} = 0{\cdot}84 \text{ tons.}$$

The total of 6·04 tons is the load per linear foot on the foundation. The maximum compressive stress in the staves is $\frac{6{\cdot}04 \times 2240}{12 \times 2{\cdot}5} = 447$ lb. per square inch.

In addition, there are compressive and tensile stresses due to the bending moment of 200 ft.-lb. caused by the lateral pressure of the grain. A rectangular section 12 in. by 2½ in. deep, if made of a homogeneous material having the same strength in compression as in tension, has a bending moment M of $f \times \frac{bh^2}{6}$ or $f = \frac{6M}{bh^2}$. Substituting the safe value of M from (4), $f = \frac{6 \times 200 \times 12}{12 \times 2{\cdot}5^2} = 192$ lb. per square inch. Thus in this case the compressive and tensile stresses are under 200 lb. per square inch. In unreinforced concrete the compressive strength greatly exceeds the tensile strength by an unpredictable amount, hence the need to base the design upon experimental data. Actually the compressive stress of 447 lb. per square inch produces compression in the ring of staves as if they were pre-stressed, and so makes them better able to resist bending. In practice, therefore, the staves are able to resist a greater bending moment than the tests indicate. Also, the total vertical compression on the staves is less than the safe permissible stress.

The weight of wheat when the silo is full is

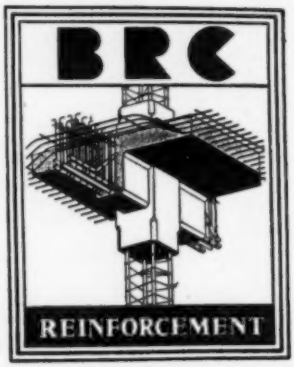
$$\frac{\pi D^2 Hw}{4 \times 2240} = \frac{\pi \times 20^2 \times 60 \times 55}{4 \times 2240} = 57{\cdot}7 \text{ tons,}$$

of which $5{\cdot}2 \times \pi \times 20 = 36{\cdot}2$ tons is carried by the wall, leaving 21·5 tons as the total load on the floor. This load is not evenly distributed, being zero near the walls and a maximum at the centre. Usually the largest silos are built on reinforced concrete rafts which are thickened under the wall.

Acknowledgment is made to the Sturtevant Engineering Co., Ltd., for information about the pneumatic conveying and conditioning plant upon which *Fig.* 2 is based, and to the U.S. Department of Agriculture for the data given in *Table* 1.

A British Standard Code for In-situ Flooring.

BRITISH Standard Code No. 204 (1951), "In-situ Flooring" (price 7*s.* 6*d.* from the British Standards Institution) deals with floor finishes of several types including granolithic and other concrete finishes, terrazzo, mastic-asphalt, pitch-mastic, cement-rubber-latex, and magnesium oxychloride. Selection of the materials, conditions suitable for the use of each type, and matters affecting design are considered, and recommendations are made regarding the bases upon which the finishes should be laid. Instructions are given on laying the finishes, and notes are included on slipperiness, colour, texture, prevention of cracking, surface treatment, and maintenance.



Professional Announcements.

Mr. D. A. Stewart, A.M.Inst.C.E., A.M.I.E.E., is now acting for Messrs. Sandberg, of 40 Grosvenor Gardens, London, S.W.1, as their consulting civil engineer, with particular reference to concrete design and testing.

Mr. P. B. Steer has joined Messrs. Stroyer & Adcock, consulting engineers, of 122 Wilton Road, London, S.W.1, as a partner.

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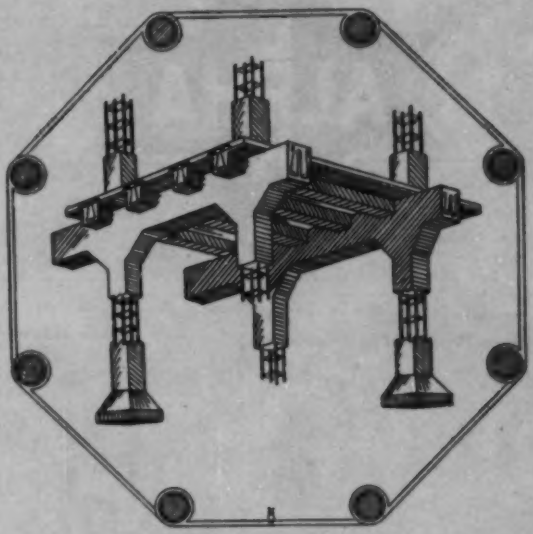

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